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	Engineering and Design ROCK FOUNDATIONS	
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30 November 1994

US Army Corps
of Engineers

ENGINEERING AND DESIGN

Rock Foundations

ENGINEER MANUAL

DEPARTMENT OF THE ARMY
U.S. Army Corps of Engineers
Washington, DC 20314-1000

CECW-EG

Manual
No. 1110-1-2908

30 November 1994

Engineering and Design
ROCK FOUNDATIONS

1. Purpose. This manual provides technical criteria and guidance for design of rock foundations for civil works or similar large military structures.

2. Applicability. This manual applies to HQUSACE elements, major subordinate commands, districts, laboratories, and field operating activities.

3. General. The manual provides a minimum standard to be used for planning a satisfactory rock foundation design for the usual situation. Unusual or special site, loading, or operating conditions may warrant sophisticated analytical designs that are beyond the scope of this manual.

FOR THE COMMANDER:

WILLIAM D. BROWN
Colonel, Corps of Engineers
Chief of Staff

CECW-EG

DEPARTMENT OF THE ARMY
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CONVERSION FACTORS, U.S. CUSTOMARY TO SI UNITS OF MEASUREMENT

U.S. customary units of measurement used in this report can be converted to SI units as follows:

Page No.	Multiply	By	To Obtain
18	cubic feet	0.02831685	cubic metres
66	degrees (angle)	0.01745329	radians
67	feet	0.3048	metres
70	gallons (U.S. liquid)	3.785412	cubic decimetres
71	inches	2.54	centimetres
72	miles (U.S. statute)	0.609347	kilometres
73	pints (U.S. liquid)	0.0004731765	cubic metres
74	pound (force)	4.448222	newtons
	pounds (force per square inch	6894.757	pascals
76	square feet	0.09290304	square metres

Chapter 1
Introduction

1-1. Purpose

This manual provides technical criteria and guidance for design of rock foundations for civil works or similar large military structures.

1-2. Applicability

This manual applies to HQUSACE elements, major subordinate commands, districts, laboratories, and field operating activities.

1-3. References

References pertaining to this manual are listed in Appendix A. References further explain or supplement a subject covered in the body of this manual. The references provided are essential publications to the users of this manual. Each reference is identified in the text by either the designated publication number or by author and date. References to cited material in tables and figures are also identified throughout the manual.

1-4. Scope of Manual

The manual provides a minimum standard to be used for planning a satisfactory rock foundation design for the usual situation. Chapter 2 provides a discussion on design considerations and factor of safety. Chapter 3 provides guidance on site investigation techniques and procedures. Chapter 4 provides guidance on rock mass characterization and classification schemes. Chapters 5 and 6 provide guidance on related topic areas of foundation deformation and settlement and foundation bearing capacity, respectively. Chapters 7 and 8 provide guidance on the sliding stability assessment of gravity structures and slopes cut into rock mass, respectively. Chapter 9 provides guidance on the design of rock anchorage systems. Chapter 10 provides guidance on selection of appropriate geotechnical instrumentation. Chapters 11 and 12 provide discussion on construction considerations and special topics, respectively. Unusual or special site, loading, or operating conditions may warrant sophisticated analytical designs that are beyond the scope of this manual.

1-5. Coordination

A fully coordinated team of geotechnical and structural engineers and engineering geologists should insure that the result of the analyses are fully integrated into the overall design feature being considered. Some of the critical aspects of the design process which require coordination are the following.

a. Details and estimates. Exploration details and preliminary estimates of geotechnical parameters, subsurface conditions and design options.

b. Features. Selection of loading conditions, loading effects, potential failure mechanisms and other related features of the analytical model.

c. Feasibility. Evaluation of the technical and economic feasibility of alternative structures.

d. Refinement of design. Refinement of the preliminary design configuration and proportions to reflect consistently the results of more detailed geotechnical site explorations, laboratory testing, and numerical analyses.

e. Unexpected variations. Modifications to features during construction due to unexpected variations in the foundation conditions.

Chapter 2
Design Considerations

2-1. Design Approach

This manual is intended to provide, where possible, a guided approach for the design of rock foundations. The concept of guided design provides for a stepped procedure for solving engineering problems that requires solution by decision making and judgment. Any design which involves rock masses requires a decision making process in which information must be obtained, considered, and reconciled before decisions and judgments can be made and supported. As such, the manual provides a stepped procedure for planning, collecting, and characterizing the information required to make intelligent decisions and value judgments concerning subsurface conditions, properties, and behavior. A fully coordinated team of geotechnical and structural engineers and engineering geologists are required to insure that rock foundation conditions and design are properly integrated into the overall design of the structure and that the completed final design of the structure is safe, efficient, and economical. Foundation characterization and design work should be guided by appropriate principles of rock mechanics.

2-2. Types of Structures

The types of structures that require analyses as described herein include concrete gravity dams, concrete retaining walls, navigation locks, embankment dams, and similar civil works or military type structures founded on rock. Although directed toward concrete structures, parts of this manual are applicable to all rock foundations.

2-3. Design Considerations

The design of rock foundations includes two usual analyses, bearing capacity and settlement analyses and sliding stability analyses. Bearing capacity and settlement analyses involve the ability of the rock foundation to support the imposed loads without bearing capacity failure and without excessive or intolerable deformations or settlements. Sliding stability analyses involve the ability of the rock foundation or slope to resist the imposed loads without shearing or sliding. Both analyses must be coordinated and satisfied in a complete design. Basic data that should be obtained during the design stage include strike, dip, thickness, continuity, and composition of all faults and shears in the foundation; depth of overburden; ground water condition; depth of weathering throughout the foundation; joint orientation and continuity; lithology; physical and engineering properties of the rock mass; and loading conditions. Potential failure modes and mechanisms must be determined. For foundation sliding stability, an adequate assessment of the stress conditions and sliding stability of the rock foundation must account for the basic behavior of the structure, the mechanism of transmitting loads to the foundation, the reaction of the foundation to the imposed loads and the effects of the foundation behavior on the structure. In addition to the above, the analyses of rock foundations must include an evaluation of the effects of seepage and of grouting performed to reduce seepage and the seepage effects. These evaluations are particularly important as related to assessment of hydraulic structures. Because of the difficulty in determining bedrock seepage, seepage paths, and the effectiveness of grouting, conservative assumptions should be used in these evaluations. For a discussion of grouting, see EM 1110-2-3504.

2-4. Factor of Safety

The factor of safety is defined in the manual in terms of the strength parameters of the rock mass. For analyses involving shear or sliding failures, the safety factor is defined as the factor by which the design shear strength must be reduced in order to bring the sliding mass into a state of limiting equilibrium along a given slip plane. This definition pertains to the shear resistance along a given slip surface. The derivation of limit equilibrium equations used to assess sliding stability involve converting stresses to forces. The equations satisfy force equilibrium for the limiting case. For analyses involving bearing capacity failures, the safety factor is defined as the ratio of allowable stress to the actual working stress. The safety factors described in the manual represent the minimum allowable safety factors to be used in the design of rock slopes and foundations for applicable structures. The minimum allowable safety factors described in this manual assume that a complete and comprehensive geotechnical investigation program has been performed. Safety factors greater than the described minimums may be warranted if uncertainties exist in the subsurface conditions or if reliable design parameters cannot be determined. Higher safety factors may also be warranted if unusual or extreme loading or operating conditions are imposed on the structure or substructure. Any relaxation of the minimum values involving rock foundations will be subject to the approval of CECW-EG and CECW-ED and should be justified by extensive geotechnical studies of such a nature as to reduce geotechnical uncertainties to a minimum.

Chapter 3
Site Investigations

3-1. Scope

This chapter describes general guidance for site investigation methods and techniques used to obtain information in support of final site evaluation, design, construction, and instrumentation phases of a project with respect to rock foundations. Once a site (preliminary or final) has been selected, the problem usually consists of adapting all phases of the project to existing terrain and rock mass conditions. Because terrain and rock mass conditions are seldom similar between project sites, it is difficult, if not impractical, to establish standardized methodologies for site investigations. In this respect, the scope of investigation should be based on an assessment of geologic structural complexity, imposed or existing loads acting on the foundation, and to some extent the consequence should a failure occur. For example, the extent of the investigation could vary from a limited effort where the foundation rock is massive and strong to extensive and detailed where the rock mass is highly fractured and contains weak shear zones. It must be recognized, however, that, even in the former case, a certain minimum of investigation is necessary to determine that weak zones are not present in the foundation. In many cases, the extent of the required field site investigation can be judged from an assessment of preliminary site studies.

3-2. Applicable Manuals

Methods and techniques commonly used in site investigations are discussed and described in other design manuals. Two manuals of particular importance are EM 1110-1-1804 and EM 1110-1-1802. It is not the intent of this manual to duplicate material discussed in existing manuals. However, discussions provided in EM 1110-1-1804 and EM 1110-1-1802 apply to both soil and rock. In this respect, this manual will briefly summarize those methods and techniques available for investigating project sites with rock foundations.

Section I
Preliminary Studies

3-3. General

Prior to implementing a detailed site investigation program, certain types of preliminary information will have been developed. The type and extent of information depends on the cost and complexity of the project. The information is developed from a thorough survey of existing information and field reconnaissance. Information on topography, geology and potential geologic hazards, surface and ground-water hydrology, seismology, and rock mass characteristics are reviewed to determine the following:

- Adequacy of available data.
- Type and extent of additional data that will be needed.
- The need for initiating critical long-term studies, such as ground water and seismicity studies, that require advance planning and early action.
- Possible locations and type of geologic features that might control the design of project features.

3-4. Map Studies

Various types of published maps can provide an excellent source of geologic information to develop the regional geology and geological models of potential or final sites. The types of available maps and their uses are described by Thompson (1979) and summarized in EM 1110-1-1804. EM 1110-1-1804 also provides sources for obtaining published maps.

3-5. Other Sources of Information

Geotechnical information and data pertinent to the project can frequently be obtained from a careful search of federal, state, or local governments as well as private industry in the vicinity. Consultation with private geotechnical engineering firms, mining companies, well drilling and development companies and state and private university staff can sometimes provide a wealth of information. EM 1110-1-1804 provides a detailed listing of potential sources of information.

3-6. Field Reconnaissance

After a complete review of available geotechnical data, a geologic field reconnaissance should be made to gather information that can be obtained without subsurface exploration. The primary objective of this initial field reconnaissance is to, insofar as possible, confirm, correct or expand geologic and hydrologic information collected from preliminary office studies. If rock outcrops are present, the initial field reconnaissance offers an opportunity to collect preliminary information on rock mass conditions that might influence the design and construction of

project features. Notation should be made of the strike and dip of major joint sets, joint spacing, joint conditions (i.e. weathering, joint wall roughness, joint tightness, joint infillings, and shear zones), and joint continuity. EM 1110-1-1804, Murphy (1985), and Chapter 4 of this manual provide guidance as to special geologic features as well as hydrologic and cultural features which should also be noted.

Section II
Field Investigations

3-7. General

This section briefly discusses those considerations necessary for completion of a successful field investigation program. The majorities of these considerations are discussed in detail in EM 1110-1-1804 and in Chapter 4 of this manual. In this respect, the minimum components that should be considered include geologic mapping, geophysical exploration, borings, exploratory excavations, and insitu testing. The focus of geologic data to be obtained will evolve as site characteristics are ascertained.

3-8. Geologic Mapping

In general, geologic mapping progresses from the preliminary studies phase with collection of existing maps and information to detailed site-specific construction mapping. Types of maps progress from areal mapping to site mapping to construction (foundation specific) mapping.

a. Areal mapping. An areal map should consist of sufficient area to include the project site(s) as well as the surrounding area that could influence or could be influenced by the project. The area and the degree of detail mapped can vary widely depending on the type and size of project and on the geologic conditions. Geologic features and information of importance to rock foundations that are to be mapped include:

(1) Faults, joints, shear zones, stratigraphy.

(2) Ground-water levels, springs, surface water or other evidence of the ground-water regime.

(3) Potential cavities due to karstic formations, mines, and tunnels.

(4) Potential problem rocks subject to dissolving, swelling, shrinking, and/or erosion.

(5) Potential rock slope instability.

(6) Gas, water, and sewer pipe lines as well as other utilities.

b. Site mapping. Site maps should be large-scaled with detailed geologic information of specific sites of interest within the project area to include proposed structure areas. Detailed description of the geologic features of existing rock foundation materials and overburden materials is essential in site mapping and subsequent explorations. The determination and description of the subsurface features must involve the coordinated and cooperative efforts of all geotechnical professionals responsible for the project design and construction.

c. Construction mapping. During construction, it is essential to map the "as built" geologic foundation conditions as accurately as possible. The final mapping is usually accomplished after the foundation has been cleaned up and just prior to the placement of concrete or backfill. Accurate location of foundation details is necessary. Permanent and easily identified planes of reference should be used. The system of measurement should tie to, or incorporate, any new or existing structure resting on the rock foundation. Foundation mapping should also include a comprehensive photographic record. A foundation map and photographic record will be made for the entire rock foundation and will be incorporated into the foundation report (ER 1110-1-1801). These maps and photographs have proved to be valuable where there were contractor claims, where future modifications to the project became necessary, or where correction of a malfunction or distress of the operational structure requires detailed knowledge of foundation conditions.

3-9. Geophysical Explorations

Geophysical techniques consist of making indirect measurements on the ground surface, or in boreholes, to obtain generalized subsurface information. Geologic information is obtained through analysis or interpretation of these measurements. Boreholes or other subsurface explorations are needed for reference and control when geophysical methods are used. Geophysical explorations are of greatest value when performed early in the field exploration program in combination with limited subsurface explorations. The explorations are appropriate for a rapid, though approximate, location and correlation of geologic features such as stratigraphy, lithology, discontinuities, ground water, and for the in-situ measurement of dynamic elastic moduli and rock densities. The cost of geophysical explorations is generally low compared with the cost of core borings or test pits, and considerable savings may be realized by judicious use of these

methods. The application, advantages, and limitations of selected geophysical methods are summarized in EM 1110-1-1804. EM 1110-1-1802 provides detailed guidance on the use and interpretation of surface and subsurface methods.

3-10. Borings

Borings, in most cases, provide the only viable exploratory tool that directly reveals geologic evidence of the subsurface site conditions. In addition to exploring geologic stratigraphy and structure, borings are necessary to obtain samples for laboratory engineering property tests. Borings are also frequently made for other uses to include collection of ground-water data, perform in-situ tests, install instruments, and explore the condition of existing structures. Boring methods, techniques, and applications are described in EM 1110-1-1804 and EM 1110-2-1907. Of the various boring methods, rock core borings are the most useful in rock foundation investigations.

a. Rock core boring. Rock core boring is the process in which diamond or other types of core drill bits are used to drill exploratory holes and retrieve rock core. If properly performed, rock core can provide an almost continuous column of rock that reflects actual rock mass conditions. Good rock core retrieval with a minimum of disturbance requires the expertise of an experienced drill crew.

(1) Standard sizes and notations of diamond core drill bits are summarized in EM 1110-1-1804. Core bits that produce 2.0 inch (nominal) diameter core (i.e., NW or NQ bit sizes) are satisfactory for most exploration work in good rock as well as provide sufficient size samples for most rock index tests such as unconfined compression, density, and petrographic analysis. However, the use of larger diameter core bits ranging from 4.0 to 6.0 inches (nominal) in diameter are frequently required to produce good core in soft, weak and/or fractured strata. The larger diameter cores are also more desirable for samples from which rock strength test specimens are prepared; particularly strengths of natural discontinuities.

(2) While the majorities of rock core borings are drilled vertically, inclined borings and in some cases oriented cores are required to adequately define stratification and jointing. Inclined borings should be used to investigate steeply inclined jointing in abutments and valley sections for dams, along spillway and tunnel alignments, and in foundations of all structures. In near vertical bedding, inclined borings can be used to reduce the total number of borings needed to obtain core samples of all strata. Where precise geological structure is required from core samples, techniques involving oriented cores are sometimes employed. In these procedures, the core is scribed or engraved with a special drilling tool so that its orientation is preserved. In this manner, both the dip and strike of any joint, bedding plane, or other planar surface can be ascertained.

(3) The number of borings and the depths to which bore holes should be advanced are dependent upon the subsurface geological conditions, the project site areas, types of projects and structural features. Where rock mass conditions are known to be massive and of excellent quality, the number and depth of boring can be minimal. Where the foundation rock is suspected to be highly variable and weak, such as karstic limestone or sedimentary rock containing weak and compressible seams, one or more boring for each major load bearing foundation element may be required. In cases where structural loads may cause excessive deformation, at least one of the boreholes should be extended to a depth equivalent to an elevation where the structure imposed stress acting within the foundation material is no more than 10 percent of the maximum stress applied by the foundation. Techniques for estimating structure induced stresses with depth are discussed in Chapter 5 of this manual.

(4) Core logging and appropriate descriptors describing the rock provide a permanent record of the rock mass conditions. Core logging procedures and appropriate rock descriptors are discussed in EM 1110-1-1804, ER 1110-1-1802, Murphy (1985), and Chapter 4 of this manual. Examples of core logs are provided in Appendix D of EM 1110-1-1804. A color photographic record of all core samples should be made in accordance with ER 1110-1-1802.

(5) The sidewalls of the borehole from which the core has been extracted offer a unique picture of the subsurface where all structural features of the rock formation are still in their original position. This view of the rock can be important when portions of rock core have been lost during the drilling operation, particularly weak seam fillers, and when the true dip and strike of the structural features are required. Borehole viewing and photography equipment include borescopes, photographic cameras, TV cameras, sonic imagery loggers, caliper loggers, and alinement survey devices. EP 1110-1-10 provides detailed information on TV and photographic systems, borescope, and televiewer. Sonic imagery and caliper loggers are discussed in detail in EM 1110-1-1802. General discussions of borehole examination techniques are also provided in EM 1110-1-1804.

b. Large-diameter borings. Large-diameter borings, 2 feet or more in diameter, are not frequently used. However, their use permits direct examination of the sidewalls of the boring or shaft and provides access for obtaining high-quality undisturbed samples. These advantages are often the principal justification for large-diameter borings. Direct inspection of the sidewalls may reveal details, such as thin weak layers or old shear planes, that may not be detected by continuous undisturbed sampling. Augers are normally used in soils and soft rock, and percussion drills, roller bits, or the calyx method are used in hard rock.

3-11. Exploratory Excavations

Test pits, test trenches, and exploratory tunnels provide access for larger-scaled observations of rock mass character, for determining top of rock profile in highly weathered rock/soil interfaces, and for some in-situ tests which cannot be executed in a smaller borehole.

a. Test pits and trenches. In weak or highly fractured rock, test pits and trenches can be constructed quickly and economically by surface-type excavation equipment. Final excavation to grade where samples are to be obtained or in-situ tests performed must be done carefully. Test pits and trenches are generally used only above the ground-water level. Exploratory trench excavations are often used in fault evaluation studies. An extension of a bedrock fault into much younger overburden materials exposed by trenching is usually considered proof of recent fault activity.

b. Exploratory tunnels. Exploratory tunnels/adits permit detailed examination of the composition and geometry of rock structures such as joints, fractures, faults, shear zones, and solution channels. They are commonly used to explore conditions at the locations of large underground excavations and the foundations and abutments of large dam projects. They are particularly appropriate in defining the extent of marginal strength rock or adverse rock structure suspected from surface mapping and boring information. For major projects where high-intensity loads will be transmitted to foundations or abutments, tunnels/adits afford the only practical means for testing inplace rock at locations and in directions corresponding to the structure loading. The detailed geology of exploratory tunnels, regardless of their purpose, should be mapped carefully. The cost of obtaining an accurate and reliable geologic map of a tunnel is usually insignificant compared with the cost of the tunnel. The geologic information gained from such mapping provides a very useful additional dimension to interpretations of rock structure deduced from other sources. A complete picture of the site geology can be achieved only when the geologic data and interpretations from surface mapping, borings, and pilot tunnels are combined and well correlated. When exploratory tunnels are strategically located, they can often be incorporated into the permanent structure. Exploratory tunnels can be used for drainage and postconstruction observations to determine seepage quantities and to confirm certain design assumptions. On some projects, exploratory tunnels may be used for permanent access or for utility conduits.

3-12. In-Situ Testing

In-situ tests are often the best means for determining the engineering properties of subsurface materials and, in some cases, may be the only way to obtain meaningful results. Table 3-1 lists in-situ tests and their purposes. In-situ rock tests are performed to determine in-situ stresses and deformation properties of the jointed rock mass, shear strength of jointed rock mass or critically weak seams within the rock mass, residual stresses within the rock mass, anchor capacities, and rock mass permeability. Large-scaled in-situ tests tend to average out the effect of complex interactions. In-situ tests in rock are frequently expensive and should be reserved for projects with large, concentrated loads. Well-conducted tests may be useful in reducing overly conservative assumptions. Such tests should be located in the same general area as a proposed structure and test loading should be applied in the same direction as the proposed structural loading. In-situ tests are discussed in greater detail in EM 1110-1-1804, the Rock Testing Handbook, and in Chapter 5 of this manual.

Section III
Laboratory Testing

3-13. General

Laboratory tests are usually performed in addition to and after field observations and tests. These tests serve to determine index values for identification and correlation, further refining the geologic model of the site and they provide values for engineering properties of the rock used in the analysis and design of foundations and cut slopes.

3-14. Selection of Samples and Tests

The selection of samples and the number and type of tests are influenced by local subsurface conditions and the size and type of structure. Prior to any laboratory testing, rock cores should have been visually classified and logged.

Table 3-1
Summary of Purpose and Type of In-Situ Tests for Rock

Purpose of Test	Type of Test
Strength	Field Vane Shear[1] Direct Shear Pressuremeter[2] Uniaxial Compressive[2] Borehole Jacking[2]
Bearing Capacity	Plate Bearing[1] Standard Penetration[1]
Stress Conditions	Hydraulic Fracturing Pressuremeter Overcoring Flat Jack Uniaxial (Tunnel) Jacking[2] Chamber (Gallery) Pressure[2]
Mass Deformability	Geophysical (Refraction)[3] Pressuremeter or Dilatometer Plate Bearing Uniaxial (Tunnel) Jacking[2] Borehole Jacking[2] Chamber (Gallery) Pressure[2]
Anchor Capacity	Anchor/Rockbolt Loading
Rock Mass Permeability	Constant Head Rising or Falling Head Well Slug Pumping Pressure Injection

Notes:
1. Primarily for clay shales, badly decomposed, or moderately soft rocks, and rock with soft seams.
2. Less frequently used.
3. Dynamic deformability.

Selection of samples and the type and number of tests can best be accomplished after development of the geologic model using results of field observations and examination of rock cores, together with other geotechnical data obtained from earlier preliminary investigations. The geologic model, in the form of profiles and sections, will change as the level of testing and the number of tests progresses. Testing requirements are also likely to change as more data become available and are reviewed for project needs. The selection of samples and type of test according to required use of the test results and geological condition is discussed in Chapter 4 of this manual. Additional guidance can be found in EM 1110-2-1902, TM 5-818-1, EM 1110-2-2909, EM 1110-1-1804, Nicholson (1983), Goodman (1976), and Hoek and Bray (1974).

3-15. Laboratory Tests

Table 3-2 summarizes laboratory tests according to purpose and type. The tests listed are the types more commonly performed for input to rock foundation analyses and design process. Details and procedures for individual test types are provided in the Rock Testing Handbook. Laboratory rock testing is discussed in Chapter 4 of this manual and in EM 1110-1-1804.

Table 3-2
Summary of Purpose and Type of In-Situ Tests for Rock

Purpose of Test	Type of Test
Strength	Uniaxial Compression Direct Shear Triaxial Compression Direct Tension Brazilian Split Point Load[1]
Deformability	Uniaxial Compression Triaxial Compress Swell Creep
Permeability	Gas Permeability
Characterization	Water Content Porosity Density (Unit Weight) Specific Gravity Absorption Rebound Sonic Velocities Abrasion Resistance

Notes:
1. Point load tests are also frequently performed in the field.

Chapter 4
Rock Mass Characterization

4-1. Scope

This chapter provides guidance in the description and engineering classification of intact rock and rock masses, the types, applications and analyses of rock property tests, the evaluation of intact rock and rock mass properties, and the selection of design parameters for project structures founded on rock. Rock mass characterization refers to the compilation of information and data to build a complete conceptual model of the rock foundation in which all geologic features that might control the stability of project structures, as well as the physical properties of those features, are identified and defined. The compilation of information and data is a continual process. The process starts with the preliminary site investigations and is expanded and refined during site exploration, laboratory and field testing, design analyses, construction and, in some cases, operation of the project structure. The order of information and data development generally reflects a district's approach to the process but usually evolves from generalized information to the specific details required by the design process. Furthermore, the level of detail required is dependent upon the project structure and the rock mass foundation conditions. For these reasons, this chapter is subdivided into five topic areas according to types of information rather than according to a sequence of tasks. Topic areas include geologic descriptions, engineering classification, shear strength parameters, bearing capacity parameters, and deformation and settlement parameters. The five topic areas provide required input to the analytical design processes described in Chapters 5, 6, 7, and 8.

4-2. Intact Rock versus Rock Mass

The in-situ rock, or rock mass, is comprised of intact blocks of rock separated by discontinuities such as joints, bedding planes, folds, sheared zones and faults. These rock blocks may vary from fresh and unaltered rock to badly decomposed and disintegrated rock. Under applied stress, the rock mass behavior is generally governed by the interaction of the intact rock blocks with the discontinuities. For purposes of design analyses, behavioral mechanisms may be assumed as discontinuous (e.g. sliding stability) or continuous (e.g. deformation and settlement).

Section I
Geologic Descriptions

4-3. General

Geologic descriptions contain some especially important qualitative and quantitative descriptive elements for intact rock and rock masses. Such descriptors are used primarily for geologic classification, correlation of stratigraphic units, and foundation characterization. A detailed description of the foundation rock, its structure, and the condition of its discontinuities can provide valuable insights into potential rock mass behavior. Geologic descriptors can, for convenience of discussion, be divided into two groups: descriptors commonly used to describe rock core obtained during site exploration core boring and supplemental descriptors required for a complete description of the rock mass. Descriptive elements are often tailored to specific geologic conditions of interest. In addition to general geologic descriptors, a number of rock index tests are frequently used to aid in geologic classification and characterization.

4-4. Rock Core Descriptors

Rock core descriptors refer to the description of apparent characteristics resulting from a visual and physical inspection of rock core. Rock core descriptors are recorded on the drilling log (ENG Form 1836) either graphically or by written description. Descriptions are required for the intact blocks of rock, the rock mass structure (i.e., fractures and bedding) as well as the condition and type of discontinuity. Criteria for the majorities of these descriptive elements are contained in Table B-2 of EM 1110-1-1804, Table 3-5 of EM 1110-1-1806, and Murphy (1985). Table 4-1 summarizes, consolidates, and, in some instances, expands descriptor criterion contained in the above references. Figures D-6 and D-7 of EM 1110-1-1804 provide examples of typical rock core logs. The following discussions provide a brief summary of the engineering significance associated with the more important descriptors.

a. Unit designation. Unit designation is usually an informal name assigned to a rock unit that does not necessarily have a relationship to stratigraphic rank (e.g. Miami oolite or Chattanooga shale).

Table 4-1
Summary of Rock Descriptors

1. Intact Blocks of Rock

a. Degree of Weathering.

(1) Unweathered: No evidence of any chemical or mechanical alteration.

(2) Slightly weathered: Slight discoloration on surface, slight alteration along discontinuities, less than 10 percent of the rock volume altered.

(3) Moderately weathered: Discoloring evident, surface pitted and altered with alteration penetrating well below rock surfaces, weathering "halos" evident, 10 to 50 percent of the rock altered.

(4) Highly weathered: Entire mass discolored, alteracation pervading nearly all of the rock with some pockets of slightly weathered rock noticeable, some minerals leached away.

(5) Decomposed: Rock reduced to a soil with relicit rock texture, generally molded and crumbled by hand.

b. Hardness.

(1) Very soft: Can be deformed by hand.

(2) Soft: Can be scratched with a fingernail.

(3) Moderately hard: Can be scratched easily with a knife.

(4) Hard: Can be scratched with difficulty with a knife.

(5) Very hard: Cannot be scratched with a knife.

c. Texture.

(1) Sedimentary rocks:

Texture	Grain Diameter	Particle Name	Rock Name
*	80 mm	cobble	conglomerate
*	5 - 80 mm	gravel	
Coarse grained	2 - 5 mm		
Medium grained	0.4 - 2 mm	sand	sandstone
Fine grained	0.1 - 0.4 mm		
Very fine grained	0.1 mm	clay, silt	shale, claystone, siltstone

* Use clay-sand texture to describe conglomerate matrix.

(2) Igneous and metamorphic rocks:

Texture	Grain Diameter
Coarse grained	5 mm
Medium grained	1 - 5 mm
Fine grained	0.1 - 1 mm
Aphanite	0.1 mm

(Continued)

Table 4-1. (Continued)

(3) Textural adjectives: Use simple standard textural adjectives such as prophyritic, vesicular, pegmatitic, granular, and grains well developed, but not sophisticated terms such as holohyaline, hypidimorphic granular, crystal loblastic, and cataclastic.

d. Lithology Macro Description of Mineral Components.

Use standard adjectives such as shaly, sandy, silty, and calcareous. Note inclusions, concretions, nodules, etc.

2. Rock Structure

a. Thickness of Bedding.

(1) Massive: 3-ft thick or greater.

(2) Thick bedded: beds from 1- to 3-ft thick.

(3) Medium bedded: beds from 4 in. to 1-ft thick.

(4) Thin bedded: 4-in. thick or less.

b. Degree of Fracturing (Jointing).

(1) Unfractured: fracture spacing - 6 ft or more.

(2) Slightly fractured: fracture spacing - 2 to 6 ft.

(3) Moderately fractured: fracture spacing - 8 in. to 2 ft.

(4) Highly fractured: fracture spacing - 2 in. to 8 in.

(5) Intensely fractured: fracture spacing - 2 in. or less.

c. Dip of Bed or Fracture.

(1) Flat: 0 to 20 degrees.

(2) Dipping: 20 to 45 degrees.

(3) Steeply dipping: 45 to 90 degrees.

3. Discontinuities

a. Joints.

(1) Type: Type of joint if it can be readily determined (i.e., bedding, cleavage, foliation, schistosity, or extension).

(2) Degree of joint wall weathering:

(i) Unweathered: No visible signs are noted of weathering; joint wall rock is fresh, crystal bright.

(ii) Slightly weathered joints: Discontinuities are stained or discolored and may contain a thin coating of altered material. Discoloration may extend into the rock from the discontinuity surfaces to a distance of up to 20 percent of the discontinuity spacing.

(iii) Moderately weathered joints: Slight discoloration extends from discontinuity planes for greater than 20 percent of the discontinuity spacing. Discontinuities may contain filling of altered material. Partial opening of grain boundaries may be observed.

(Continued)

Table 4-1. (Concluded)

(iv) Highly weathered joints: same as Item 1.a.(4).

(v) Completely weathered joints: same as Item 1.a.(5).

(3) Joint wall separations: General description of separation it it can be estimated from rock core; open or closed; if open note magnitude; filled or clean.

(4) Roughness:

(i) Very rough: Near vertical ridges occur on the discontinuity surface.

(ii) Rough: Some ridges are evident; asperities are clearly visible and discontinuity surface feels very abrasive.

(iii) Slighlty rough: Asperities on the discontinuity surface are distinguishable and can be felt.

(iv) Smooth: Surface appears smooth and feels so to the touch.

(v) Slickensided: Visual evidence of polishing exists.

(5) Infilling: Source, type, and thickness of infilling; alterated rock, or by deposition; clay, silt, etc.; how thick is the filler.

b. Faults and Shear Zones.

(1) Extent: Single plane or zone; how thick.

(2) Character: Crushed rock, gouge, clay infilling, slickensides.

b. Rock type. Rock type refers to the general geologic classification of the rock (e.g. basalt, sandstone, limestone, etc.). Certain physical characteristics are ascribed to a particular rock type with a geological name given according to the rocks mode of origin. Although the rock type is used primarily for identification and correlation, the type is often an important preliminary indicator of rock mass behavior.

c. Degree of weathering. The engineering properties of a rock can be, and often are, altered to varying degrees by weathering of the rock material. Weathering, which is disintegration and decomposition of the in-situ rock, is generally depth controlled, that is, the degree of weathering decreases with increasing depth below the surface.

d. Hardness. Hardness is a fundamental characteristic used for classification and correlation of geologic units. Hardness is an indicator of intact rock strength and deformability.

e. Texture. The strength of an intact rock is frequently affected, in part, by the individual grains comprising the rock.

f. Structure. Rock structure descriptions describe the frequency of discontinuity spacing and thickness of bedding. Rock mass strength and deformability are both influenced by the degree of fracturing.

g. Condition of discontinuities. Failure of a rock mass seldom occurs through intact rock but rather along discontinuities. The shear strength along a joint is dependent upon the joint aperture presence or absence of filling materials, the type of the filling material and roughness of the joint surface walls, and pore pressure conditions.

h. Color. The color of a rock type is used not only for identification and correlation, but also for an index of rock properties. Color may be indicative of the mineral constituents of the rock or of the type and degree of weathering that the rock has undergone.

i. Alteration. The rock may undergo alteration by geologic processes at depth, which is distinctively different from the weathering type of alteration near the surface.

4-5. Supplemental Descriptors

Descriptors and descriptor criterion discussed in paragraph 4-4 and summarized in Table 4-1 can be readily obtained from observation and inspection of rock core. However, certain important additional descriptors cannot be obtained from core alone. These additional descriptors include orientation of discontinuities, actual thicknesses of discontinuities, first-order roughness of discontinuities, continuity of discontinuities, cavity details, and slake durability.

a. Orientation of discontinuities. Because discontinuities represent directional planes of weakness, the orientation of the discontinuity is an important consideration in assessing sliding stability and, to some extent, bearing capacity and deformation/settlement. Retrieved core, oriented with respect to vertical and magnetic north, provides a means for determining discontinuity orientation. A number of manufacturers market devices for this purpose. However, most of these techniques abound with practical difficulties (e.g. see Hoek and Bray 1974). The sidewalls of the borehole from which conventional core has been extracted offer a unique picture of the subsurface where all structural features of the rock mass are still in their original position. In this respect, techniques that provide images of the borehole sidewalls such as the borehole camera, the borescope, TV camera or sonic imagery (discussed in Chapter 3, EM 1110-1-1804, EP 1110-1-10, and EM 1110-1-1802) offer an ideal means of determining the strike and dip angles of discontinuities. The orientation of the discontinuity should be recorded on a borehole photo log. The poles of the planes defined by the strike and dip angles of the discontinuities should then be plotted on an equal area stereonet. Equal area stereonet pole plots permit a statistical evaluation of discontinuity groupings or sets, thus establishing likely bounds of strike and dip orientations. A stereographic projection plot should then be made of the bounding discontinuity planes for each set of discontinuities to assess those planes which are kinematically free to slide. Goodman (1976), Hoek and Bray (1974), and Priest (1985) offer guidance for stereonet pole plots and stereographic projection techniques.

b. Discontinuity thickness. The drilling and retrieving of a rock core frequently disturb the discontinuity surfaces. For this reason, aperture measurements of discontinuity surfaces obtained from rock core can be misleading. The best source for joint aperture information is from direct measurement of borehole surface images (e.g. borehole photographs and TV camera recordings). The actual aperture measurement should be recorded on a borehole photo log. An alternative to recording actual measurements is to describe aperture according to the following descriptors:

(1) Very tight: separations of less than 0.1 mm.

(2) Tight: separations between 0.1 and 0.5 mm.

(3) Moderately open: separations between 0.5 and 2.5 mm.

(4) Open: separations between 2.5 and 10 mm.

(5) Very wide: separations between 10 and 25 mm.

For separations greater than 25 mm the discontinuity should be described as a major discontinuity.

c. First-order roughness of discontinuities. First-order roughness refers to the overall, or large scale, asperities along a discontinuity surface. Figure 4-1 illustrates the difference between first-order large scale asperities and the smaller, second-order asperities commonly associated with roughnesses representative of the rock core scale. The first-order roughness is generally the major contributor to shear strength development along a discontinuity (see paragraph 4-14*b* below for further discussion). A description of this large scale roughness can only be evaluated from an inspection of exposed discontinuity traces or surfaces. An inspection of rock outcrops in the vicinity of the project site offers an inexpensive means of obtaining this information. Critically oriented joint sets, for which outcrops are not available, may require excavation of inspection adits or trenches. Descriptors such as stepped, undulating, or planar should be used to describe noncritical surfaces. For critically oriented discontinuities, the angles of inclination, (referred to as the *i* angle) between the average dip of the discontinuity and first-order asperities should be measured and recorded

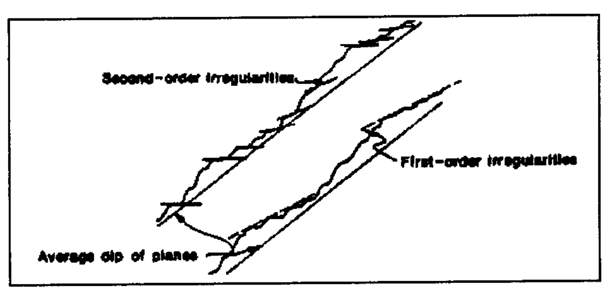


Figure 4-1. Rough discontinuity surface with first-order and second-order asperities (after Patton and Deere 1970)

(Figure 4-1). Hoek and Bray (1974) provide guidance for measuring first-order asperity angles.

d. Continuity of discontinuities. The continuity of a joint influences the extent to which the intact rock material and the discontinuities separately affect the behavior of the rock mass. In essence, the continuity, or lack of continuity, determines whether the strength that controls the stability of a given structure is representative of a discontinuous rock surface or a combination of discontinuous surfaces and intact rock. For the case of retaining structures, such as gravity dams and lockwalls, a discontinuity is considered fully continuous if its length is greater than the base width in the direction of potential sliding.

e. Cavities. Standard rock coring procedures are capable of detecting the presence of cavities as well as their extent along the borehole axis. However, an evaluation of the volumetric dimensions requires three-dimensional inspection. Downhole TV cameras, with their relatively long focal lengths, provide a means for inspecting cavities. Rock formations particularly susceptible to solutioning (e.g. karstic limestone, gypsum, and anhydrite) may require excavation of inspection trenches or adits to adequately define the location and extent of major cavities. A description of a cavity should include its geometric dimensions, the orientation of any elongated features, and the extent of any infilling as well as the type of infilling material.

4-6. Index Tests

Intact samples of rock may be selected for index testing to further aid in geological classification and as indicators of rock mass behavior. As a matter of routine, certain tests will always be performed on representative cores from each major lithological unit and/or weathered class. The number of tests should be sufficient to characterize the range of properties. Routine tests include water content, unit weight, and unconfined compression tests. Additional tests for durability, tensile strength, specific gravity, absorption, pulse velocity, and ultrasonic elastic constants and permeability tests as well as a petrographic examination may be dictated by the nature of the rock or by the project requirements. Types of classification and index tests which are frequently used for rock are listed in Table 4-2.

Section II
Rock Mass Classification

4-7. General

Following an appropriate amount of site investigation the rock mass can be divided or classified into zones or masses of similar expected performance. The similar performance may be excavatability, strength, deformability, or any other characteristic of interest, and is determined by use of all of the investigative tools previously described. A good rock mass classification system will:

- Divide a particular rock mass into groups of similar behavior.
- Provide a basis for understanding the characteristics of each group.
- Facilitate planning and design by yielding quantitative data required for the solution of real engineering problems.
- Provide a common basis for effective communication among all persons concerned with a given project.

A meaningful and useful rock mass classification system must be clear and concise, using widely accepted terminology. Only the most significant properties, based on measured parameters that can be derived quickly and inexpensively, should be included. The classification should be general enough that it can be used for a tunnel, slope, or foundation. Because each feature of a rock mass (i.e. discontinuities, intact rock, weathering, etc.) has a different significance, a ranking of combined factors is necessary to satisfactorily describe a rock mass. Each project may need site-specific zoning or rock mass classification, or it may benefit from use of one of the popular existing systems.

4-8. Available Classification Systems

Numerous rock mass classification systems have been developed for universal use. However, six have enjoyed greater use. The six systems include Terzaghi's Rock Load Height Classification (Terzaghi 1946); Lauffer's

Table 4-2
Laboratory Classification and Index Tests for Rock

Test	Test Method	Remarks
Unconfined (uniaxial) compression	RTH[1] 111	Primary index test for strength and deformability of intact rock; required input to rock mass classification systems.
Point load test	RTH 325	Indirect method to determine unconfined compressive (UC) strength; can be performed in the field on core pieces unsuitable for UC tests.
Water content	RTH 106	Indirect indication of porosity of intact rock or clay content of sedimentary rock.
Unit weight and total porosity	RTH 109	Indirect indication of weathering and soundness.
Splitting strength of rock (Brazilian tensile strength method)	RTH 113	Indirect method to determine the tensile strength of intact rock.
Durability	ASTM[2] D-4644	Index of weatherability of rock exposed in excavations.
Specific gravity of solids	RTH 108	Indirect indication of soundness of rock intended for use as riprap and drainage aggregate.
Pulse velocities and elastic constants	RTH 110	Index of compressional wave velocity and ultrasonic elastic constants for correlation with in-situ geophysical test results.
Rebound number	RTH 105	Index of relative hardness of intact rock cores.
Permeability	RTH 114	Intact rock (no joints or major defects).
Petrographic examination	RTH 102	Performed on representative cores of each significant lithologic unit.
Specific gravity and absorption	RTH 107	Indirect indication of soundness and deformability

Notes:
1. Rock Testing Handbook.
2. American Society for Testing and Materials.

Classification (Lauffer 1958); Deere's Rock Quality Designation (RQD) (Deere 1964); RSR Concept (Wickham, Tiedemann, and Skinner 1972); Geomechanics System (Bieniawski 1973); and the Q-System (Barton, Lien, and Lunde 1974). Most of the above systems were primarily developed for the design of underground excavations. However, three of the above six classification systems have been used extensively in correlation with parameters applicable to the design of rock foundations. These three classification systems are the Rock Quality Designation, Geomechanics System, and the Q-System.

4-9. Rock Quality Designation

Deere (1964) proposed a quantitative index obtained directly from measurements of rock core pieces. This index, referred to as the Rock Quality Designation (RQD), is defined as the ratio (in percent) of the total length of sound core pieces 4 in. (10.16 cm) in length or longer to the length of the core run. The RQD value, then, is a measure of the degree of fracturing, and, since the ratio counts only sound pieces of intact rock, weathering is accounted for indirectly. Deere (1964) proposed the

following relationship between the RQD index and the engineering quality of the rock mass. The determination of RQD during core recovery is simple and straightforward. The RQD index is internationally recognized

RQD, percent	Rock Quality
< 25	Very poor
25 < 50	Poor
50 < 75	Fair
75 < 90	Good
90 < 100	Excellent

as an indicator of rock mass conditions and is a necessary input parameter for the Geomechanic System and Q-System. Since core logs should reflect to the maximum extent possible the rock mass conditions encountered, RQD should be determined in the field and recorded on the core logs. Deere and Deere (1989) provides the latest guidance for determining RQD.

4-10. Geomechanics Classification

a. General. The Geomechanics Classification, or Rock Mass Rating (RMR) system, proposed by Bieniawski (1973), was initially developed for tunnels. In recent years, it has been applied to the preliminary design of rock slopes and foundations as well as for estimating the in-situ modulus of deformation and rock mass strength. The RMR uses six parameters that are readily determined in the field:

- Uniaxial compressive strength of the intact rock.
- Rock Quality Designation (RQD).
- Spacing of discontinuities.
- Condition of discontinuities.
- Ground water conditions.
- Orientation of discontinuities.

All but the intact rock strength are normally determined in the standard geological investigations and are entered on an input data sheet (see Table B-1, Appendix B). The uniaxial compressive strength of rock is determined in accordance with standard laboratory procedures but can be readily estimated on site from the point-load strength index (see Table 4-2).

b. Basic RMR determination. The input data sheet (Table B-1, Appendix B) summarizes, for each core hole, all six input parameters. The first five parameters (i.e. strength, RQD, joint spacing, joint conditions, and ground water) are used to determine the basic RMR. Importance ratings are assigned to each of the five parameters in accordance with Part A of Table B-2, Appendix B. In assigning the rating for each core hole, the average conditions rather than the worst are considered. The importance ratings given for joint spacings apply to rock masses having three sets of joints. Consequently, a conservative assessment is obtained when only two sets of discontinuities are present. The basic rock mass rating is obtained by adding up the five parameters listed in Part A of Table B-2, Appendix B.

c. Adjustment for discontinuity orientation. Adjustment of the basic RMR value is required to include the effect of the strike and dip of discontinuities. The adjustment factor (a negative number) and hence the final RMR value, will vary depending upon the engineering application and the orientation of the structure with respect to the orientation of the discontinuities. The adjusted values, summarized in Part B of Table B-2, Appendix B, are divided into five groups according to orientations which range from very favorable to very unfavorable. The determination of the degree of favorability is made by reference to Table B-3 for assessment of discontinuity orientation in relation to dams (Part A), and tunnels (Part B).

d. Rock mass class. After the adjustment is made in accordance with Part B, Table B-2, Appendix B, the rock mass ratings are placed in one of five rock mass classes in Part C, Table B-2, Appendix B. Finally, the ratings are grouped in Part D of Table B-2, Appendix B. This section gives the practical meaning of each rock class, and a qualitative description is provided for each of the five rock mass classes. These descriptions range from "very good rock" for class I (RMR range from 81 to 100) to "very poor rock" for class V (RMR ranges < 20). This classification also provides a range of cohesion values and friction angles for the rock mass.

4-11. Q-System

The Q-system, proposed by Barton, Lien, and Lunde (1974) was developed specifically for the design of tunnel support systems. As in the case of the Geomechanics System, the Q-system has been expanded to provide preliminary estimates. Likewise, the Q-system incorporates

the following six parameters and the equation for obtaining rock mass quality Q:

- Rock Quality Designation (RQD).
- Number of discontinuity sets.
- Roughness of the most unfavorable discontinuity.
- Degree of alteration or filling along the weakest discontinuity.
- Water inflow.
- Stress condition.

$$Q = (\mathrm{RQD}/J_n) \times (J_r/J_a) \times (J_w/\mathrm{SRF}) \tag{4-1}$$

where

RQD = Rock Quality Designation

J_n = joint set number

J_r = joint roughness number

J_a = joint alteration number

J_w = joint water reduction number

SRF = stress reduction number

Table B-4, Appendix B, provides the necessary guidance for assigning values to the six parameters. Depending on the six assigned parameter values reflecting the rock mass quality, Q can vary between 0.001 to 1000. Rock quality is divided into nine classes ranging from exceptionally poor (Q ranging from 0.001 to 0.01) to exceptionally good (Q ranging from 400 to 1000).

4-12. Value of Classification Systems

There is perhaps no engineering discipline that relies more heavily on engineering judgment than rock mechanics. This judgment factor is, in part, due to the difficulty in testing specimens of sufficient scale to be representative of rock mass behavior and, in part, due to the natural variability of rock masses. In this respect, the real value of a rock mass classification systems is appropriately summarized by Bieniawski (1979). "...no matter which classification system is used, the very process of rock mass classification enables the designer to gain a better understanding of the influence of the various geologic parameters in the overall rock mass behavior and, hence, gain a better appreciation of all the factors involved in the engineering problem. This leads to better engineering judgment. Consequently, it does not really matter that there is no general agreement on which rock classification system is best; it is better to try two or more systems and, through a parametric study, obtain a better "feel" for the rock mass. Rock mass classification systems do not replace site investigations, material descriptions, and geologic work-up. They are an adjunct to these items and the universal schemes, in particular, have special value in relating the rock mass in question to engineering parameters based on empirical knowledge."

Section III
Shear Strength

4-13. General

The shear strength that can be developed to resist sliding in a rock foundation or a rock slope is generally controlled by natural planes of discontinuity rather than the intact rock strength. The possible exception to this rule may include structures founded on, or slopes excavated in, weak rock or where a potential failure surface is defined by planes of discontinuities interrupted by segments of intact rock blocks. Regardless of the mode of potential failure, the selection of shear strength parameters for use in the design process invariably involves the testing of appropriate rock specimens. Selection of the type of test best suited for intact or discontinuous rock, as well as selection of design shear strength parameters, requires an appreciation of rock failure characteristics. Discussions on rock failure characteristics are contained in TR GL-83-13 (Nicholson 1983a) and Goodman (1980).

4-14. Rock Failure Characteristics

Failure of a foundation or slope can occur through the intact rock, along discontinuities or through filling material contained between discontinuities. Each mode of failure is defined by its own failure characteristics.

a. Intact rock. At stress levels associated with low head gravity dams, retaining walls and slopes, virtually all rocks behave in a brittle manner at failure. Brittle failure is marked by a rapid increase in applied stress, with small strains, until a peak stress is obtained. Further increases in strain cause a rapid decrease in stress until the residual stress value is reached. While the residual stress value is generally unique for a given rock type and minor principal stress, the peak stress is dependent upon the size of

the specimen and the rate that the stress is applied. Failure envelopes developed from plots of shear stress versus normal stress are typically curvilinear.

b. Discontinuities. The typical failure envelope for a clean discontinuous rock is curvilinear as is intact rock. Surfaces of discontinuous rock are composed of irregularities or asperities ranging in roughness from almost smooth to sharply inclined peaks. Conceptually there are three modes of failure--asperity override at low normal stresses, failure through asperities at high normal stresses, and a combination of asperity override and failure through asperities at intermediate normal stresses. Typically, those normal stresses imposed by Corps structures are sufficiently low that the mode of failure will be controlled by asperity override. The shear strength that can be developed for the override mode is scale dependent. Initiation of shear displacement causes the override mode to shift from the small scale second-order irregularities to the large scale first-order irregularities. As indicated in Figure 4-1, first-order irregularities generally have smaller angles of inclination (*i* angles) than second-order irregularities. Shear strengths of discontinuities with rough undulating surfaces reflect the largest scale effects with small surface areas (laboratory specimen size) developing higher shear stress than large surface areas (in-situ scale). Figure 4-2 illustrates the influence of both scale effects and discontinuity surface roughnesses.

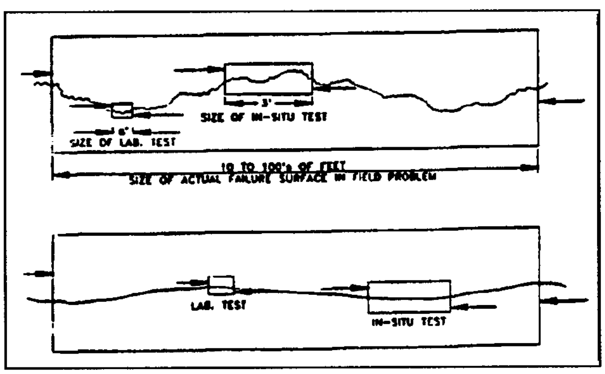


Figure 4-2. Effect of different size specimens selected along a rough and a smooth discontinuity surface (after Deere et al. 1967)

c. Filled discontinuities. Failure modes of filled discontinuities can range from those modes associated with clean unfilled discontinuities to those associated with soil. Four factors contribute to their strength behavior: thickness of the filler material, material type, stress history and displacement history.

(1) Thickness. Research indicates that the strength of discontinuities with filler thicknesses greater than two times the amplitude of the surface undulations is controlled by the strength of the filler material. In general, the thicker the filler material with respect to the amplitude of the asperities, the less the scale effects.

(2) Material type. The origin of the filler material and the strength characteristics of the joint are important indicators. Sources of filler material include products of weathering or overburden washed into open, water-conducting discontinuities; precipitation of minerals from the ground water; by-products of weathering and alterations along joint walls; crushing of parent rock surfaces due to tectonic and shear displacements; and thin seams deposited during formation. In general, fine-grained clays are more frequently found as fillers and are more troublesome in terms of structural stability.

(3) Stress history. For discontinuities containing fine-grained fillers, the past stress history determines whether the filler behaves as a normally consolidated or overconsolidated soil.

(4) Displacement history. An important consideration in determining the strength of discontinuities filled with fine-grained cohesive materials is whether or not the discontinuity has been subjected to recent displacement. If significant displacement has occurred, it makes little difference whether the material is normally or overconsolidated since it will be at or near its residual strength.

4-15. Failure Criteria

a. Definition of failure. The term "failure" as applied to shear strength may be described in terms of load, stress, deformation, strain or other parameters. The failure strengths typically associated with the assessment of sliding stability are generally expressed in terms of peak, residual, ultimate or as the shear strength at a limiting strain or displacement as illustrated in Figure 4-3.

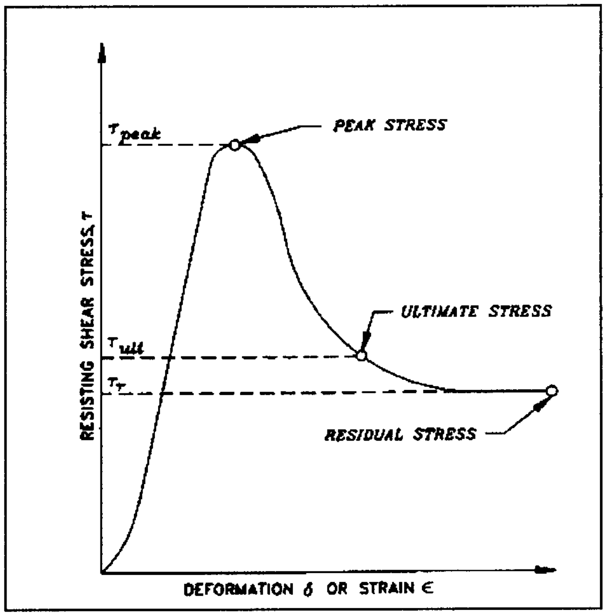


Figure 4-3. Shear test failure as defined by peak, ultimate, and residual stress levels (after Nicholson 1983a)

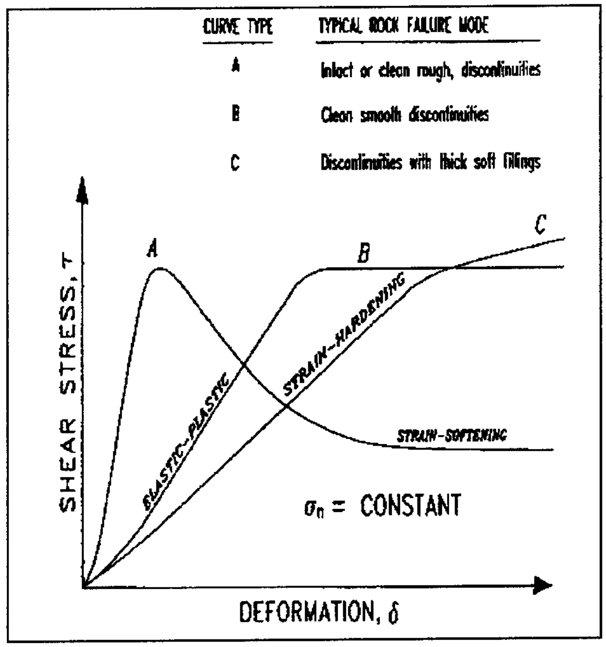


Figure 4-4. Hypothetical shear stress-deformation curves from drained direct shear tests on: (a) strain-softening; (b) elastic-plastic; and (c) strain-hardening materials (after Nicholson 1983a)

The appropriate definition of failure generally depends on the shape of the shear stress versus shear deformation/strain curve as well as the mode of potential failure. Figure 4-4 illustrates the three general shear stress versus deformation curves commonly associated with rock failure.

b. Linear criteria. Failure criteria provide an algebraic expression for relating the shear strength at failure with a mathematical model necessary for stability analysis. Mathematical limit equilibrium models used to access sliding stability incorporate linear Mohr-Coulomb failure criterion as follows:

$$\tau_f = c + \sigma_n \tan \phi \qquad (4\text{-}2)$$

where

τ_f = the shearing stress developed at failure

σ_n = stress normal to the failure plane

The c and ϕ parameters are the cohesion intercept and angle of internal friction, respectively. Figure 4-5 illustrates the criterion. It must be recognized that failure envelopes developed from shear tests on rock are generally curved. However, with proper interpretation, failure envelopes over most design stress ranges can be closely approximated by the linear Coulomb equation required by the analytical stability model.

c. Bilinear criteria. Bilinear criteria (Patton 1966; Goodman 1980) offer a more realistic representation of the shear stress that can be developed along clean (unfilled) discontinuities. These criteria divide a typical curved envelope into two linear segments as illustrated in Figure 4-6. The maximum shear strength that can be developed at failure is approximated by the following equations:

$$\tau_f = \sigma_n \tan (\phi_u + i) \qquad (4\text{-}3)$$

and

$$\tau_f = c_a + \sigma_n \tan \phi_r \qquad (4\text{-}4)$$

where

τ_f = maximum (peak) shear strength at failure

σ_n = stress normal to the shear plane (discontinuity)

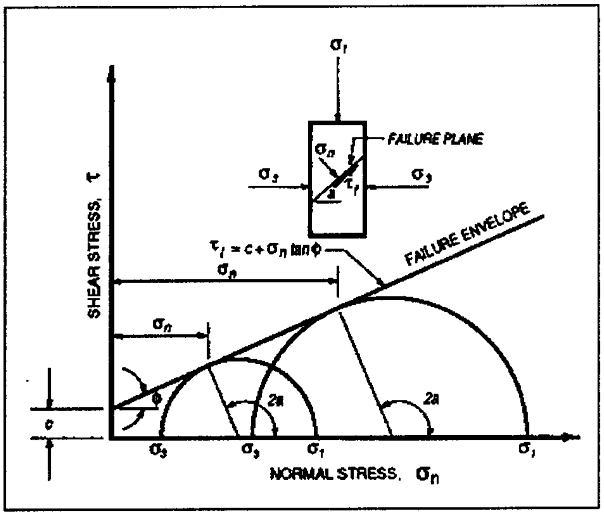


Figure 4-5. Mohr-Coulomb relationship with respect to principal stresses and shear stress

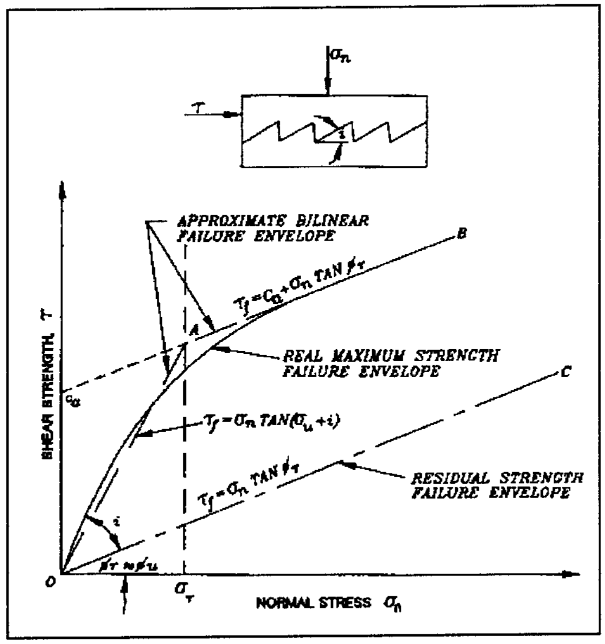


Figure 4-6. Typical approximate bilinear and real curvilinear failure envelopes for modeled discontinuous rock

ϕ_u = the basic friction angle on smooth planar sliding surface

i = angle of inclination of the first order (major) asperities

ϕ_r = the residual friction angle of the material comprising the asperities

c_a = the apparent cohesion (shear strength intercept) derived from the asperities

For unweathered discontinuity surfaces, the basic friction angle and the residual friction angle are, for practical purposes, the same. The intercept of the two equations (i.e. σ_τ in Figure 4-6) occurs at the transition stress between the modes of failure represented by asperity override and shearing of the asperities. Normal stresses imposed by Corps projects are below the transition stress (σ_τ) for the majority of rock conditions encountered. Hence, maximum shear strengths predicted by Equation 4-3, generally control design.

4-16. Shear Strength Tests

Table 4-3 lists tests that are useful for measuring the shear strength of rock. Details of the tests, test apparatus, and procedures are given in the Rock Testing Handbook (see references Table 4-3), EM 1110-1-1804, and GL-83-14 (Nicholson 1983b.).

4-17. Shear Strength Testing Program

The testing program for measuring shear strengths of rock specimens reflects the intended use of the test results (preliminary or final design), the type of specimens (intact or discontinuous), the cost, and, in some cases, the availability of testing devices. In general, the testing program closely parallels the field exploration program, advancing from preliminary testing where modes of potential failure are poorly defined to detailed testing of specific modes of potential failure controlling project design. As a minimum, the following factors should be considered prior to initiating the final detailed phase of testing: the sensitivity of stability with respect to strengths, loading conditions, suitability of tests used to model modes of failure, and the selection of appropriate test specimens.

a. Sensitivity. A sensitivity analysis should be performed to evaluate the relative sensitivity of the shear strengths required to provide an adequate calculated factor of safety along potential failure planes. Such analysis frequently indicates that conservative and inexpensively obtained strengths often provide an adequate measure of stability, without the extra cost of more precisely defined in-situ strengths.

Table 4-3
Tests to Measure Shear Strength of Rock

Test	Reference	Remarks
Laboratory direct shear	RTH 203[1]	Strength along planes of weakness (bedding), discontinuities or rock-concrete contact; not recommended for intact rock.
Laboratory triaxial	RTH 202	Deformation and strength of inclined compression planes of weakness and discontinuities; strain and strength of intact rock.
In-situ direct shear	RTH 321	Expensive; generally reserved for critically located discontinuities filled with a thin seam of very weak material.
In-situ uniaxial	RTH 324	Expensive; primarily used for defining compression scale effects of weak intact rock; several specimen sizes usually tested.

Notes:
1. Rock Testing Handbook.

b. Loading conditions. Shear tests on rock specimens should duplicate the anticipated range of normal stresses imposed by the project structure along potential failure planes. Duplication of the normal stress range is particularly important for tests on intact rock, or rough natural discontinuities, that exhibit strong curvilinear failure envelopes.

c. Shear test versus mode of failure. Both triaxial and direct shear tests are capable of providing shear strength results for all potential modes of failure. However, a particular type of test may be considered better suited for modes of failure. The suitability of test types with respect to modes of failure should be considered in specifying a testing program.

(1) Laboratory triaxial test. The triaxial compression test is primarily used to measure the undrained shear strength and in some cases the elastic properties of intact rock samples subjected to various confining pressures. By orienting planes of weakness the strength of natural joints, seams, and bedding planes can also be measured. The oriented plane variation is particularly useful for obtaining strength information on thinly filled discontinuities containing soft material. Confining pressures tend to prevent soft fillers from squeezing out of the discontinuity. The primary disadvantage of the triaxial test is that stresses normal to the failure plane cannot be directly controlled. Since clean discontinuities are free draining, tests on clean discontinuities are considered to be drained. Tests on discontinuities filled with fine-grained materials are generally considered to be undrained (drained tests are possible but require special testing procedures). Tests on discontinuities with coarse grained fillers are generally considered to be drained. Detailed procedures for making laboratory triaxial tests are presented in the Rock Testing Handbook (RTH 204).

(2) Laboratory direct shear test. The laboratory direct shear test is primarily used to measure the shear strength, at various normal stresses, along planes of discontinuity or weakness. Although sometimes used to test intact rock, the potential for developing adverse stress concentrations and the effects from shear box induced moments makes the direct shear test less than ideally suited for testing intact specimens. Specimen drainage conditions, depending on mode of failure, are essentially the same as for laboratory triaxial tests discussed above. The test is performed on core samples ranging from 2 to 6 inches in diameter. Detailed test procedures are presented in the Rock Testing Handbook (RTH 203).

(3) In-situ direct shear test. In-situ direct shear tests are expensive and are only performed where critically located, thin, weak, continuous seams exist within relatively strong adjacent rock. In such cases, conservative lower bound estimates of shear strength seldom provide adequate assurance against instability. The relatively

large surface area tested is an attempt to address unknown scale effects. However, the question of how large a specimen is large enough still remains. The test, as performed on thin, fine-grained, clay seams, is considered to be an undrained test. Test procedure details are provided in the Rock Testing Handbook (RTH 321). Technical Report S-72-12 (Zeigler 1972) provides an indepth review of the in-situ direct shear test.

(4) In-situ uniaxial compression test. In-situ uniaxial compression tests are expensive. The test is used to measure the elastic properties and compressive strength of large volumes of virtually intact rock in an unconfined state of stress. The uniaxial strength obtained is useful in evaluating the effects of scale. However, the test is seldom performed just to evaluate scale effects on strength.

d. Selection of appropriate specimens. No other aspect of rock strength testing is more important than the selection of the test specimens that best represents the potential failure surfaces of a foundation. Engineering property tests conducted on appropriate specimens directly influence the analysis and design of projects. As a project progresses, team work between project field personnel and laboratory personnel is crucial in changing type of test, test specimen type, and number of tests when site conditions dictate. The test specimen should be grouped into rock types and subgrouped by unconfined compressive strength, hardness, texture, and structure, or any other distinguishing features of the samples. This process will help in defining a material's physical and mechanical properties. General guidance on sample selection is provided in EM 1110-1-1804. However, shear strength is highly dependent upon the mode of failure, i.e. intact rock, clean discontinuous rock, and discontinuities containing fillers. Furthermore, it must be realized that each mode of failure is scale dependent. In this respect, the selection of appropriate test specimens is central to the process of selecting design shear strength parameters.

4-18. Selection of Design Shear Strength Parameters

a. Evaluation procedures. The rock mass within a particular site is subject to variations in lithology, geologic structure, and the in-situ stress. Regardless of attempts to sample and test specimens with flaws and/or weaknesses present in the rock mass, these attempts, at best, fall short of the goal. The number, orientation, and size relationship of the discontinuities and/or weaknesses may vary considerably, thus affecting load distribution and the final results. In addition to these factors, laboratory results are dependent on the details of the testing procedures, equipment, sampling procedures, and the condition of the sample at the time of the test. The result of these numerous variables is an expected variation in the laboratory test values which further complicates the problem of data evaluation. The conversion from laboratory measured strength parameters to in-situ strength parameters requires a careful evaluation and analysis of the geologic and laboratory test data. Also, a combination of experience and judgment is necessary to assess the degree or level of confidence that is required in the selected parameters. As a minimum, the following should be considered: the most likely mode of prototype failure, the factor of safety, the design use, the cost of tests, and the consequence of a failure. A flow diagram illustrating examples of factors to consider in assessing the level of confidence in selected design strengths is shown in Figure 4-7. In general, an increase in assessed confidence should either reflect increasing efforts to more closely define prototype shear strength, at increasing cost, or increasing conservatism in selected design strengths to account for the uncertainties of the in-situ strength.

b. Selection procedures. Failure envelopes for likely upper and lower bounds of shear strength can generally be determined for the three potential modes of failure; intact rock, clean discontinuities, and filled discontinuities. These limits bound the range within which the in-situ strength is likely to lie. Technical Report GL-83-13 (Nicholson 1983a) describes appropriate test methods and procedures to more accurately estimate in-situ strength parameters. Efforts to more accurately define in-situ strengths must reflect the level of confidence that is required by the design.

(1) Intact rock. Plots of shear stress versus normal stress, from shear test on intact rock, generally result in considerable data scatter. In this respect, nine or more tests are usually required to define both the upper and lower bounds of shear strength. Figure 4-8 shows a plot of shear stress versus normal stress for a series of tests on a weak limestone. Failure envelopes obtained from a least-squares best fit of upper and lower bounds, as well as all data points, are shown in Figure 4-8. Variations in cohesion values are generally greater than the variations in the friction angle values. With a sufficient number of tests to define scatter trends, over a given range of normal stresses the confidence that can be placed in the friction angle value exceeds the level of confidence that can be placed in the cohesion value. As a rule, a sufficient factor of safety can be obtained from lower bound

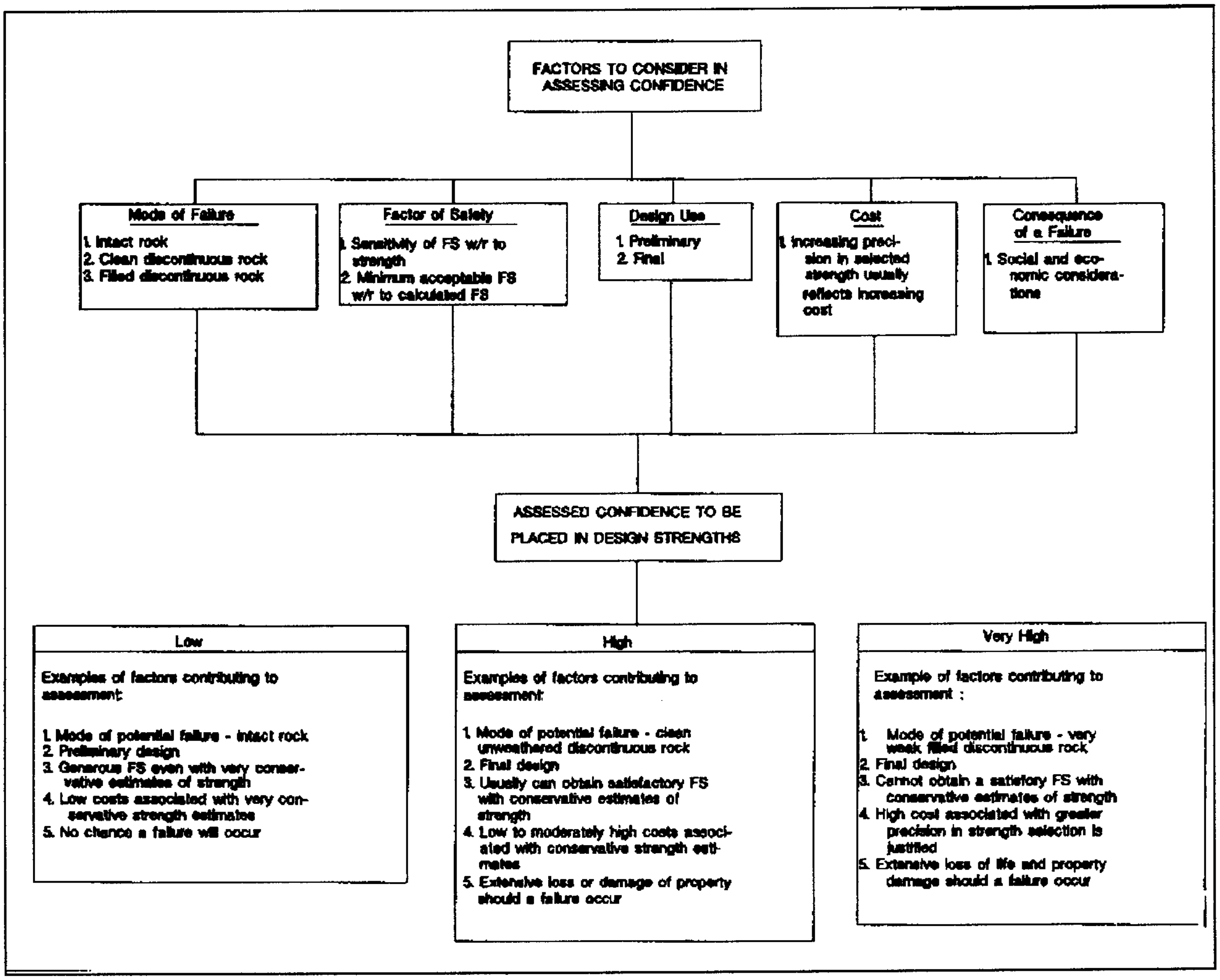


Figure 4-7. Flow diagram illustrating examples of factors to consider in assessing the confidence to be placed in selected design strengths (after Nicholson 1983a)

estimates of shear strength obtained from laboratory tests. For design cases where lower bound shear strength estimates provide marginal factors of safety, the influence of scale effects must be evaluated. Shear strengths obtained from laboratory tests on small specimens should be reduced to account for scale effects. In this respect, Pratt et al. (1972) and Hoek and Brown (1980) suggest that the full- scale uniaxial compressive strength of intact rock can be as much as 50 percent lower than the uniaxial compressive strength of a small intact laboratory specimen. In the absence of large scale tests to verify the effects of scale, conservative estimates of the shear strength parameters (cohesion and friction angle) which account for scale effects can be obtained by reducing the lower bound cohesion value by 50 percent. This reduced lower bound cohesion value is to be used with the lower bound friction angle value for marginal design cases.

(2) Clean discontinuities. Upper and lower bounds of shear strength for clean discontinuities can be obtained from laboratory tests on specimens containing natural discontinuities and presawn shear surfaces, respectively. The number of tests required to determine the bounds of strength depends upon the extent of data scatter observed in plots of shear stress versus normal stress. As a rule, rough natural discontinuity surfaces will generate more data scatter than smooth discontinuity surfaces. Hence, lower bound strengths obtained from tests on smooth sawn surfaces may require as few as three tests while upper bound strength from tests on very rough natural

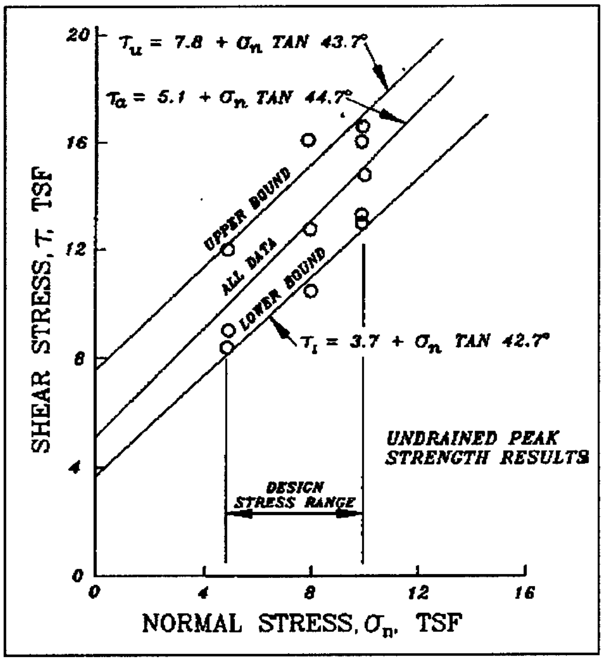


Figure 4-8. Direct shear test results on intact limestone illustrating upper and lower bounds of data scatter

engineering judgment can not be overly emphasized. discontinuity surfaces may require nine or more tests. Data scatter and/or curvilinear trends in plots of shear stress versus normal stress may result in cohesion intercepts. In such cases, cohesion intercepts are ignored in the selection of design shear strengths. The lower bound failure envelope obtained from shear tests on smooth sawn surfaces defines the basic friction angle (ϕ_u in Equation 4-3). The friction angle selected for design may be obtained from the sum of the basic friction angle and an angle representative of the effective angle of inclination (i in Equation 4-3) for the first-order asperities. The sum of the two angles must not exceed the friction angle obtained from the upper bound shear tests on natural discontinuities. The primary difficulty in selecting design friction values lies in the selection of an appropriate i angle. Discontinuity surfaces or outcrop traces of discontinuities are not frequently available from which to base a reasonable estimate of first order inclination angles. In such cases estimates of the i angle must rely on sound engineering judgment and extensive experience in similar geology.

(3) Filled discontinuities. In view of the wide variety of filler materials, previous stress and displacement histories and discontinuity thicknesses, standardization of a procedure for selecting design shear strengths representative of filled discontinuities is difficult. The process is further complicated by the difficulty in retrieving quality specimens that are representative of the discontinuity in question. For these reasons, the use of sound Uncertainties associated with unknown conditions effecting shear strength must be reflected in increased conservatism. Generally, the scale effects associated with discontinuous rock are lessened as the filler material becomes thicker in relation to the amplitude of the first-order joint surface undulations. However, potential contributions of the first-order asperities to the shear strength of a filled joint are, as a rule, not considered in the strength selection process because of the difficulty in assessing their effects. Shear strengths that are selected based on in-situ direct shear test of critically located weak discontinuities are the exception to this general rule, but there still remains the problem of appropriate specimen size. As illustrated in Figure 4-9, the displacement history of the discontinuity is of primary concern. If a filled discontinuity has experienced recent displacement, as evident by the presence of slicken-sides, gouge, mismatched joint surfaces, or other features, the strength representative of the joint is at or near its residual value. In such cases, shear strength selection should be based on laboratory residual shear tests of the natural joint. Possible cohesion intercepts observed from the test results should not be included in the selection of design strengths. If the discontinuity has not experienced previous displacement, the shear strength is at or near its peak value. Therefore, whether the filler material is normally or overconsolidated is of considerable importance. In this respect, the shear stress level used to define failure of laboratory test specimens is dependent upon the material properties of the filler. The following definitions of failure stress are offered as general guidance to be tempered with sound engineering judgment: peak strength should be used for filler consisting of normally consolidated cohesive materials and all cohesionless materials; peak or ultimate strength is used for filler consisting of overconsolidated cohesive material of low plasticity; ultimate strength, peak strength of remolded filler, or residual strength is used (depending on material characteristics) for filler consisting of overconsolidated cohesive material of medium to high plasticity.

(4) Combined modes. Combined modes of failure refer to those modes in which the critical failure path is defined by segments of both discontinuous planes and planes passing through intact rock. Selection of appropriate shear strengths for this mode of failure is

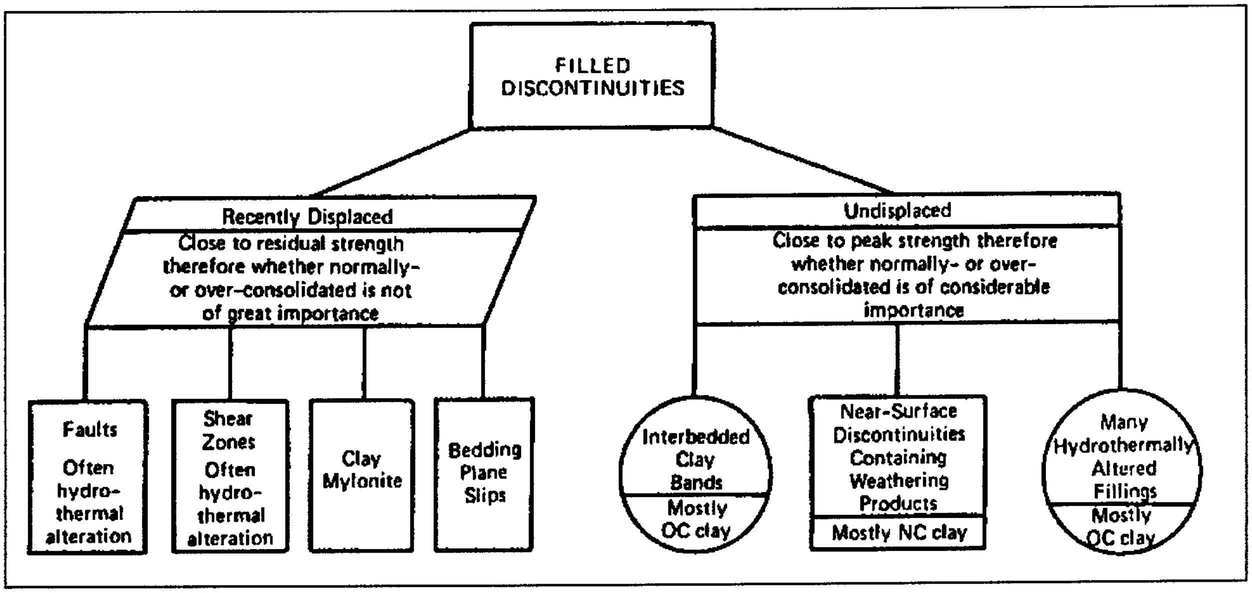


Figure 4-9. Simplified division of filled discontinuities into displaced and undisplaced and normally consolidated (NC) and overconsolidated (OC) categories (after Barton 1974)

particularly difficult for two reasons. First, the precentages of the failure path defined by discontinuities or intact rock are seldom known. Second, strains/displacements necessary to cause failure of intact rock are typically an order of magnitude (a factor of 10) smaller than those displacements associated with discontinuous rock. Hence, peak strengths of the intact rock proportion will already have been mobilized and will likely be approaching their residual strength before peak strengths along the discontinuities can be mobilized. For these reasons, selection of appropriate strengths must be based on sound engineering judgment and experience gained from similar projects constructed in similar geological conditions. Shear strength parameters selected for design must reflect the uncertainties associated with rock mass conditions along potential failure paths as well as mechanisms of potential failure (i.e. sliding along discontinuities versus shear through intact rock).

Section IV
Deformation and Settlement

4-19. General

The deformational response of a rock mass is important in seismic analyses of dams and other large structures as well as the static design of gravity and arch dams, tunnels, and certain military projects. Analytical solutions for deformation and settlement of rock foundations are invariably based on the assumption that the rock mass behaves as a continuum. As such, analytical methods used to compute deformations and the resulting settlements are founded on the theory of elasticity. The selection of design parameters, therefore, involves the selection of appropriate elastic properties: Poisson's ratio and the elastic modulus. Although it is generally recognized that the Poisson's ratio for a rock mass is scale and stress dependent, a unique value is frequently assumed. For most rock masses, Poisson's ratio is between 0.10 and 0.35. As a rule, a poorer quality rock mass has a lower Poisson's ratio than good quality rock. Hence, the Poisson's ratio for a highly fractured rock mass may be assumed as 0.15 while the value for a rock mass with essentially no fractures may be assumed as equal to the value of intact rock. A method for determining Poisson's ratios for intact rock core specimens is described in the Rock Testing Handbook (RTH 201). The selection of an appropriate elastic modulus is the most important parameter in reliable analytical predictions of deformation and settlement. Rock masses seldom behave as an ideal elastic material. Furthermore, modulus is both scale and stress dependent. As a result, stress-strain responses typical of a rock mass are not linear. The remaining parts of this section will address appropriate definitions of modulus, scale effects, available methods for estimating modulus values and the selection of design values.

4-20. Moduli Definitions

The elastic modulus relates the change in applied stress to the change in the resulting strain. Mathematically, it is expressed as the slope of a given stress-strain response. Since a rock mass seldom behaves as an ideal linear elastic material, the modulus value is dependent upon the proportion of the stress-strain response considered. Figure 4-10 shows a stress-strain curve typical of an in-situ rock mass containing discontinuities with the various moduli that can be obtained. Although the curve, as shown, is representative of a jointed mass, the curve is also typical of intact rock except that upper part of the curve tends to be concaved downward at stress levels approaching failure. As can be seen in Figure 4-10 there are at least four portions of the stress-strain curve used for determining in-situ rock mass moduli: the initial tangent modulus, the elastic modulus, the tangent recovery modulus, and the modulus of deformation.

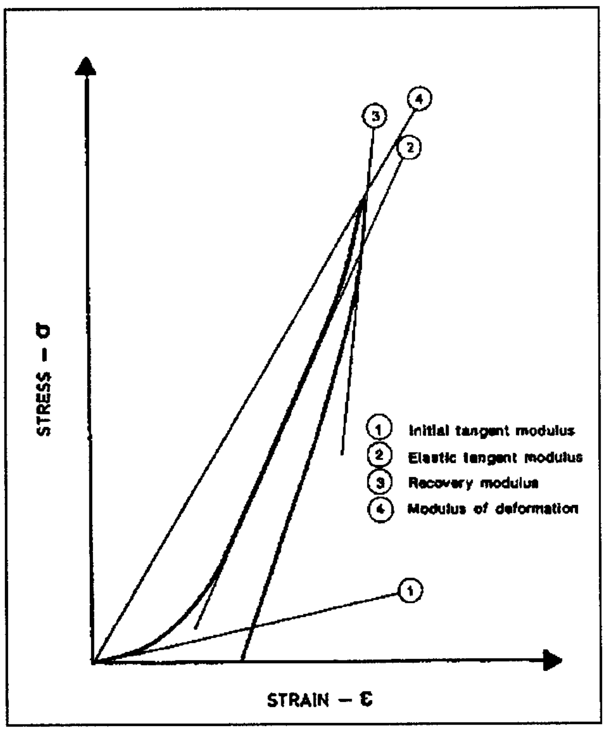


Figure 4-10. Stress-strain curve typical of in-situ rock mass with various moduli that can be obtained

a. Initial tangent modulus. The initial tangent modulus is determined from the slope of a line constructed tangent to the initial concave upward section of the stress-strain curve (i.e. line 1 in Figure 4-10). The initial curved section reflects the effects of discontinuity closure in in-situ tests and micro-crack closure in tests on small laboratory specimens.

b. Elastic modulus. Upon closure of discontinuities/micro-cracks, the stress-strain becomes essentially linear. The elastic modulus, frequently referred to as the modulus of elasticity, is derived from the slope of this linear (or near linear) portion of the curve (i.e. line 2 in Figure 4-10). In some cases, the elastic modulus is derived from the slope of a line constructed tangent to the stress-strain curve at some specified stress level. The stress level is usually specified as 50 percent of the maximum or peak stress.

c. Recovery modulus. The recovery modulus is obtained from the slope of a line constructed tangent to the initial segment of the unloading stress-strain curve (i.e. line 3 in Figure 4-10). As such, the recovery modulus is primarily derived from in-situ tests where test specimens are seldom stressed to failure.

d. Modulus of deformation. Each of the above moduli is confined to specific regions of the stress-strain curve. The modulus of deformation is determined from the slope of the secant line established between zero and some specified stress level (i.e. line 4 in Figure 4-10). The stress level is usually specified as the maximum or peak stress.

4-21. Test Methods for Estimating Modulus

There are at least nine different test methods available to estimate rock modulus. While all nine methods have been used in estimating modulus for design purpose, only the following seven have been standardized: the uniaxial compression tests; uniaxial-jacking tests; the pressure-meter test; plate load test; pressure-chamber tests; radial-jack tests; and borehole-jacking tests. Other test methods that are not standardized but are described in the literature include flat-jack tests and tunnel-relaxation tests.

a. Uniaxial compression tests. Laboratory uniaxial compression tests are the most frequently used tests for estimating rock modulus. These tests are performed on relatively small, intact, specimens devoid of discontinuities. As such, the results obtained from these tests over estimate the modulus values required for design analyses. Laboratory tests are useful in that the derived moduli provide an upper limit estimate. In-situ uniaxial compression tests are capable of testing specimens of sufficient size to contain a representative number of discontinuities. Modulus values obtained from in-situ tests are considered to be more reliable. This test method is more versatile

than some in-situ methods in that test specimens can be developed from any exposed surface. However, the tests are expensive. The Rock Testing Handbook describes test procedures for both laboratory (RTH 201) and in-situ (RTH 324) uniaxial compression tests for the estimation of modulus.

b. Uniaxial jacking tests. The uniaxial jack test involves the controlled loading and unloading of opposing rock surfaces developed in a test adit or trench. The loads are applied by means of large hydraulic jacks which react against two opposing bearing pads. Measurement of the rock mass deformational response below the bearing pads provides two sets of data from which moduli can be derived. The test is expensive. However, the majority of the expense is associated with the excavation of the necessary test adit or trench. The test procedures are described in the Rock Testing Handbook (RTH 365).

c. Pressure meter tests. The pressure meter test expands a fluid filled flexible membrane in a borehole causing the surrounding wall of rock to deform. The fluid pressure and the volume of fluid equivalent to the volume of displaced rock are recorded. From the theory of elasticity, pressure and volume changes are related to the modulus. The primary advantage of the pressure meter is its low cost. The test is restricted to relatively soft rock. Furthermore, the test influences only a relatively small volume of rock. Hence, modulus values derived from the tests are not considered to be representative of rock mass conditions. The test procedures are described in the Rock Testing Handbook (RTH 362).

d. Plate load tests. The plate load test is essentially the same as the uniaxial jacking test except that only one surface is generally monitored for deformation. If sufficient reaction such as grouted cables can be provided, the test may be performed on any rock surface. Details of the test procedures are discussed in the Rock Testing Handbook (RTH 364-89).

e. Flat-jack tests. The flat-jack test is a simple test in which flat-jacks are inserted into a slot cut into a rock surface. Deformation of the rock mass caused by pressurizing the flat-jack is measured by the volumetric change in the jack fluid. The modulus is derived from relationships between jack pressure and deformation. However, analysis of the test results is complicated by boundary conditions imposed by the test configuration. The primary advantages of the test lie in its ability to load a large volume of rock and its relatively low cost. The test procedures are described by Lama and Vutukuri (1978).

f. Pressure-chamber tests. Pressure-chamber tests are performed in large, underground openings. Generally, these openings are test excavations such as exploratory tunnels or adits. Pre-existing openings, such as caves or mine chambers, can be used if available and applicable to project conditions. The opening is lined with an impermeable membrane and subjected to hydraulic pressure. Instrumented diametrical gages are used to record changes in tunnel diameter as the pressure load increases. The test is usually performed through several load-unload cycles. The data are subsequently analyzed to develop load-deformation curves from which a modulus can be obtained. The test is capable of loading a large volume of a rock mass from which a representative modulus can be obtained. The test, however, is extremely expensive. The test procedures are described in the Rock Testing Handbook (RTH-361).

g. Radial jacking tests. Radial jacking test is a modification of the pressure chamber test where pressure is applied through a series of jacks placed close to each other. While the jacking system varies, the most common system consists of a series of flat-jacks sandwiched between steel rings and the tunnel walls. The Rock Testing Handbook (RTH-367) describes the test procedures.

h. Borehole-jacking tests. Instead of applying a uniform pressure to the full cross-section of a borehole as in pressuremeter tests, the borehole-jack presses plates against the borehole walls using hydraulic pistons, wedges, or flatjacks. The technique allows the application of significantly higher pressures required to deform hard rock. The Goodman Jack is the best known device for this test. The test is inexpensive. However, the test influences only a small volume of rock and theoretical problems associated with stress distribution at the plate/rock interface can lead to problems in interpretation of the test results. For these reasons, the borehole-jacking tests are considered to be index tests rather than tests from which design moduli values can be estimated. The tests are described in the Rock Testing Handbook (RTH-368).

i. Tunnel relaxation tests. Tunnel relaxation tests involve the measurement of wall rock deformations caused by redistribution of in-situ stresses during tunnel excavation. Except for a few symmetrically shaped openings with known in-situ stresses, back calculations to obtain modulus values from observed deformations generally require numerical modeling using finite element or boundary element computer codes. The high cost of the test is associated with the expense of tunnel excavation.

4-22. Other Methods for Estimating Modulus

In addition to test methods in which modulus values are derived directly from stress-strain responses of rock, there are at least two additional methods frequently used to obtain modulus values. The two methods include seismic and empirical methods.

a. Seismic methods. Seismic methods, both downhole and surface, are used on occasion to determine the in-situ modulus of rock. The compressional wave velocity is mathematically combined with the rock's mass density to estimate a dynamic Young's modulus, and the shear wave velocity is similarly used to estimate the dynamic rigidity modulus. However, since rock particle displacement is so small and loading transitory during these seismic tests, the resulting modulus values are nearly always too high. Therefore, the seismic method is generally considered to be an index test. EM 1110-1-1802 and Goodman (1980) describe the test.

b. Empirical methods. A number of empirical methods have been developed that correlate various rock quality indices or classification systems to in-situ modulus. The more commonly used include correlations between RQD, RMR and Q.

(1) RQD correlations. Deere, Merritt, and Coon (1969) developed an empirical relationship for the in-situ modulus of deformation according to the following formula:

$$E_d = [(0.0231)(\mathrm{RQD}) - 1.32]\, E_{t50} \qquad (4\text{-}5)$$

where

E_d = in-situ modulus of deformation

RQD = Rock Quality Designation (in percent)

E_{t50} = laboratory tangent modulus at 50 percent of the unconfined compressive strength

From Equation 4-5 it can be seen that the relationship is invalid for RQD values less than approximately 60 percent. In addition, the relationship was developed from data that indicated considerable variability between in-situ modulus, RQD, and the laboratory tangent modulus.

(2) RMR correlations. A more recent correlation between in-situ modulus of deformation and the RMR Classification system was developed by Serafim and Pereira (1983) that included an earlier correlation by Bieniawski (1978).

$$E_d = 10^{\frac{\mathrm{RMR}-10}{40}}$$

where

E_d = in-situ modulus of deformation (in GPa)

RMR = Rock Mass Rating value

Equation 4-6 is based on correlations between modulus of deformation values obtained primarily from plate bearing tests conducted on rock masses of known RMR values ranging from approximately 25 to 85.

(3) Q correlations. Barton (1983) suggested the following relationships between in-situ modulus of deformation and Q values:

$$E_d\ (\text{mean}) = 25 \log Q \qquad (4\text{-}7a)$$

$$E_d\ (\text{min.}) = 10 \log Q \qquad (4\text{-}7b)$$

$$E_d\ (\text{max.}) = 40 \log Q \qquad (4\text{-}7c)$$

where

E_d (mean) = mean value of in-situ modulus of deformation (in GPa)

E_d (min.) = minimum or lower bound value of in-situ modulus of deformation (in GPa)

E_d (max.) = maximum or upper bound value of in-situ modulus of deformation (in GPa)

Q = rock mass quality value

4-23. Considerations in Selecting Design Modulus Values

Modulus values intended to be representative of in-situ rock mass conditions are subject to extreme variations. There are at least three reasons for these variations: variations in modulus definitions, variability in the methods used to estimate modulus, and rock mass variability.

a. Variations in modulus definitions. As noted in paragraph 4-20, the stress-strain responses of rock masses are not linear. Hence, modulus values used in design are

dependent upon the portion of the stress-strain curve considered. Because the modulus of deformation incorporates all of the deformation behavior occurring under a given design stress range, it is the most commonly used modulus in analytical solutions for deformation.

b. Variability in methods. Modulus values obtained from tests are not unique in that the value obtained depends, for the most part, on the test selected. There are at least two reasons for this non-uniqueness. First, with the exception of laboratory compression tests, all of the methods discussed above are in-situ tests in which modulus values are calculated from suitable linear elastic solutions or represent correlations with modulus values derived from in-situ tests. Therefore, the validity of a given method depends to some extent on how well a given solution models a particular test. Finally, the volume of rock influenced by a particular test is a significant factor in how well that test reflects in-situ behavior. Recognizing the potential variation in modulus determinations, the plate-load test has become the most commonly used test for deriving the in-situ modulus of deformation for those projects requiring confidence in estimated values representative of in-situ conditions.

c. Rock mass variability. Deformational predictions of foundation materials underlying major project structures such as gravity and arch dams may require analytical solutions for multilayer media. In this respect, the selection of appropriate design deformation moduli will require consideration of not only natural variability within rock layers but also variability between layers.

4-24. Selection of Design Moduli

As in the selection of design shear strengths, the moduli values used for design purposes are selected rather than determined. The selection process requires sound engineering judgment by an experienced team of field and office geotechnical professionals. However, unlike shear strength selection, in which both upper and lower bounds of strength can generally be defined, only the upper bound of the deformation modulus can be readily predicted. This upper bound is derived from unconfined compression tests on intact rock. In addition, the natural variability of the foundation rock as well as the variability in derived modulus values observed from available methods used to predict modulus, complicates the selection of representative values of modulus. For these reasons, the selection process should not rely on a single method for estimating modulus, but rather the selection process should involve an intergrated approach in which a number of methods are incorporated. Index tests, such as the laboratory unconfined compression test and borehole test devices (Goodman jack, pressuremeter, and dilatometers), are relatively inexpensive to perform and provide insight as to the natural variability of the rock as well as establish the likely upper bounds of the in-situ modulus of deformation. Empirical correlations between the modulus of deformation and rock mass classification systems (i.e. Equations 4-5, 4-6, and 4-7) are helpful in establishing likely ranges of in-situ modulus values and provide approximate values for preliminary design. Index testing and empirical correlations provide initial estimates of modulus values and form the bases for identifying zones of deformable foundation rock that may adversely effect the performance of project structures. Sensitivity analyses, in which initial estimates of deformation moduli are used to predict deformation response, are essential to define zones critical to design. The design of structures founded on rock judged to be critical to performance must either reflect increasing conservatism in the selected modulus of deformation values or an increase in large scale in-situ testing (i.e. plate bearing tests, etc.) to more precisely estimate in-situ moduli. The high cost of in-situ tests generally limits the number of tests that can be performed. In this respect, it may not be economically feasible to conduct tests in rock representative of all critical zones; particularly for large projects founded on highly variable rock. In such cases site-specific correlations should be developed between the modulus of deformation values derived from both borehole index tests and large scale in-situ tests and rock mass classification systems (i.e. either the RMR system or the Q-system). If care is taken in selecting test locations, such correlations provide a basis for extrapolating modulus of deformation values that are representative of a wide range of rock mass conditions.

Section V
Use of Selected Design Parameters

4-25. General

For use of the selected design parameters, refer to the appropriate chapters as follows:

a. Chapter 5 - Deformation and Settlement (modulus of deformation).

b. Chapter 6 - Bearing Capacity (shear strength).

c. Chapter 7 - Sliding Stability (shear strength).

d. Chapter 8 - Cut Slope Stability in Rock (shear strength).

e. Chapter 9 - Anchorage Systems (shear strength).

Chapter 5
Deformation and Settlement

5-1. Scope

This chapter describes the necessary elements for estimating and treating settlement, or heave, of structures that are caused by the deformation of the foundation rock. This chapter is subdivided into four sections. Topic areas for the four sections include categories of deformation, analytical methods for predicting the magnitude of deformation, estimating allowable magnitudes of deformation, and methods available for reducing the magnitude of deformation.

Section I
Categories of Rock Mass Deformation

5-2. General

Deformations that may lead to settlement or heave of structures founded on or in rock may be divided into two general categories: time-dependent deformations and time-independent deformations.

5-3. Time-Dependent Deformations

Time-dependent deformations can be divided into three different groups according to the mechanistic phenomena causing the deformation. The three groups include consolidation, swelling, and creep.

a. Consolidation. Consolidation refers to the expulsion of pore fluids from voids due to an increase in stress. As a rule, consolidation is associated with soils rather than rock masses. However, rock masses may contain fractures, shear zones, and seams filled with clay or other compressible soils. Sedimentary deposits with interbedded argillaceous rock such as shales and mud stones may also be susceptible to consolidation if subjected to sufficiently high stresses. Consolidation theory and analytical methods for predicting the magnitude of consolidation are addressed in EM 1110-1-1904 and in Instruction Report K-84-7 (Templeton 1984).

b. Swelling. Certain expansive minerals, such as montmorillonite and anhydrite, react and swell in contact with water. Upon drying, these minerals are also susceptible to shrinking. The montmorillonite minerals are generally derived from alteration of ferromagnesian minerals, calcic feldspars, and volcanic rocks and are common in soils and sedimentary rocks. Anhydrite represents gypsum without its water of crystallization and is usually found as beds or seams in sedimentary rock as well as in close association with gypsum and halite in the evaporite rocks. Guidance on procedures and techniques for predicting the behavior of foundations on or in swelling minerals is contained in EM 1110-1-1904, TM 5-818-1, and Miscellaneous Paper GL-89-27 (Johnson 1989).

c. Creep. Creep refers to a process in which a rock mass continues to strain with time upon application of stress. Creep can be attributed to two different mechanisms; mass flow and propagation of microfractures. Mass flow behavior is commonly associated with certain evaporite rock types such as halite and potash. Creep associated with microfracture propagation has been observed in most rock types. Figure 5-1 shows a typical strain-time curve for various constant stress levels. As indicated in Figure 5-1, the shapes of the strain-time curve are a function of the magnitude of the applied stress. Creep will generally occur if the applied stress is within the range associated with nonstable fracture propagation. The transition between stable and nonstable fracture propagation varies, depending upon rock type, but typically is on the order of, at least, 50 percent of the uniaxial compressive strength. Most structures founded on rock generate stress levels well below the transition level. Hence, creep is generally not a problem for the majority of Corps projects. Structures founded on weak rock are the possible exceptions to this rule. Although standardized procedures are available to estimate creep properties of intact rock specimens (i.e. RTH-205) what these properties mean in terms of rock mass behavior is poorly understood. For this reason, estimates of creep response for structures founded on rock masses require specialized studies and, in some cases, research.

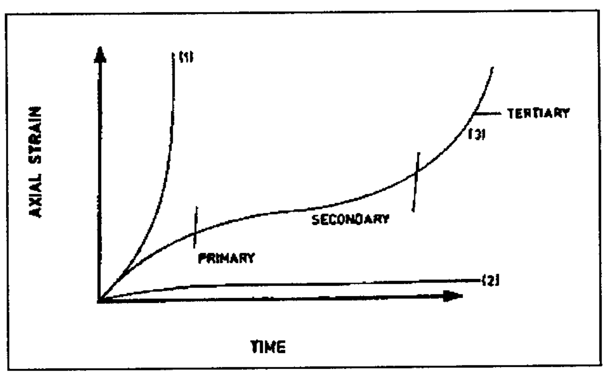


Figure 5-1. Postulated strain-time curves at (1) very high maintained stress levels, (2) moderate maintained stress levels, and (3) high maintained stress levels (from Farmer 1983)

5-4. Time-Independent Deformations

Time-independent deformations refer to those deformations which are mechanistically independent of time. Time-independent deformations include deformations generated by prefailure elastic strains, post-failure plastic strains, and deformations resulting from large shear induced or rotational displacements. Prudent foundation designs preclude consideration of post-failure behavior. Hence, time-independent deformations, as relating to foundation design, refer to deformations that occur as a result of prefailure elastic strains. Analytical methods for estimating rock mass deformations discussed in Section II of this chapter pertain to elastic solutions.

Section II
Analytical Methods

5-5. General

Analytical methods for calculating deformations of foundations may be divided into two general groups, closed form mathematical models and numerical models. The choice of a method in design use depends on how well a particular method models the design problem, the availability, extent, and precision of geological and structural input parameters, the intended use of calculated deformations (i.e. preliminary or final design), and the required accuracy of the calculated values.

5-6. Closed Form Methods

Closed form methods refer to explicit mathematical equations developed from the theory of elasticity. These equations are used to solve for stresses and strains/deformations within the foundation rock as a function of structure geometry, load and rigidity and the elastic properties of the foundation rock. Necessary simplifying assumptions associated with the theory of elasticity impose certain limitations on the applicability of these solutions. The most restrictive of these assumptions is that the rock is assumed to be homogeneous, isotropic and linearly elastic. Poulos and Davis (1974) provide a comprehensive listing of equations, tables, and charts to solve for stresses and displacements in soils and rock. Complex loadings and foundation shapes are handled by superposition in which complex loads or shapes are reduced to a series of simple loads and shapes. Conditions of anisotropy, stratification, and inhomogeneity are treated with conditional assumptions. If sound engineering judgment is exercised to insure that restrictive and conditional assumptions do not violate reasonable approximations of prototype conditions, closed form solutions offer reasonable predictions of performance.

a. Input parameters. Closed form solutions require, as input parameters, the modulus of elasticity and Poisson's ratio. For estimates of deformation/settlement in rock, the modulus of deformation, E_d, is used in place of modulus of elasticity. Techniques for estimating the modulus of deformation are described in Chapter 4 of this manual. Poisson's ratio typically varies over a small range from 0.1 to 0.35. Generally, the ratio values decrease with decreasing rock mass quality. Because of the small range of likely values and because solutions for deformation are relatively insensitive to assigned values, Poisson's ratio is usually assumed.

b. Depth of influence. Stresses within the foundation rock that are a result of foundation loads decrease with depth. In cases where the foundation is underlain by multi-layered rock masses, with each layer having different elastic properties, the depth of influence of the structural load must be considered. For the purpose of computing deformation/settlement, the depth of influence is defined as the depth at which the imposed stress acting normal to the foundation plane diminishes to 20 percent of the maximum stress applied by the foundation. If there is no distinct change in the elastic properties of the subsurface strata within this depth, elastic solutions for layered media need not be considered. Poulus and Davis (1974) and Naval Facilities Engineering Command, NAVFAC DM-7.1 (1982) provide equations and charts based on Boussinesq's equations for estimating stresses with depth imposed by various foundation shapes and loading conditions.

c. Layered foundation strata. Poulus and Davis (1974) provided procedures for estimating the deformation/settlement of foundations with the depth of influence for up to four different geologic layers. Multilayer strata, in which the ratios of moduli of deformation of any of the layers does not exceed a factor of three, may be treated as a single layer with a representative modulus of deformation equivalent to the weighted average of all layers within the depth of influence. A weighted average considers that layers closer to the foundation influence the total deformation to a greater extent than deeper layers. Figure 5-2 shows a foundation underlain by a multi-layer strata containing *n* number of layers within the depth of influence. The weighted average modulus of deformation may be obtained from Equation 5-1.

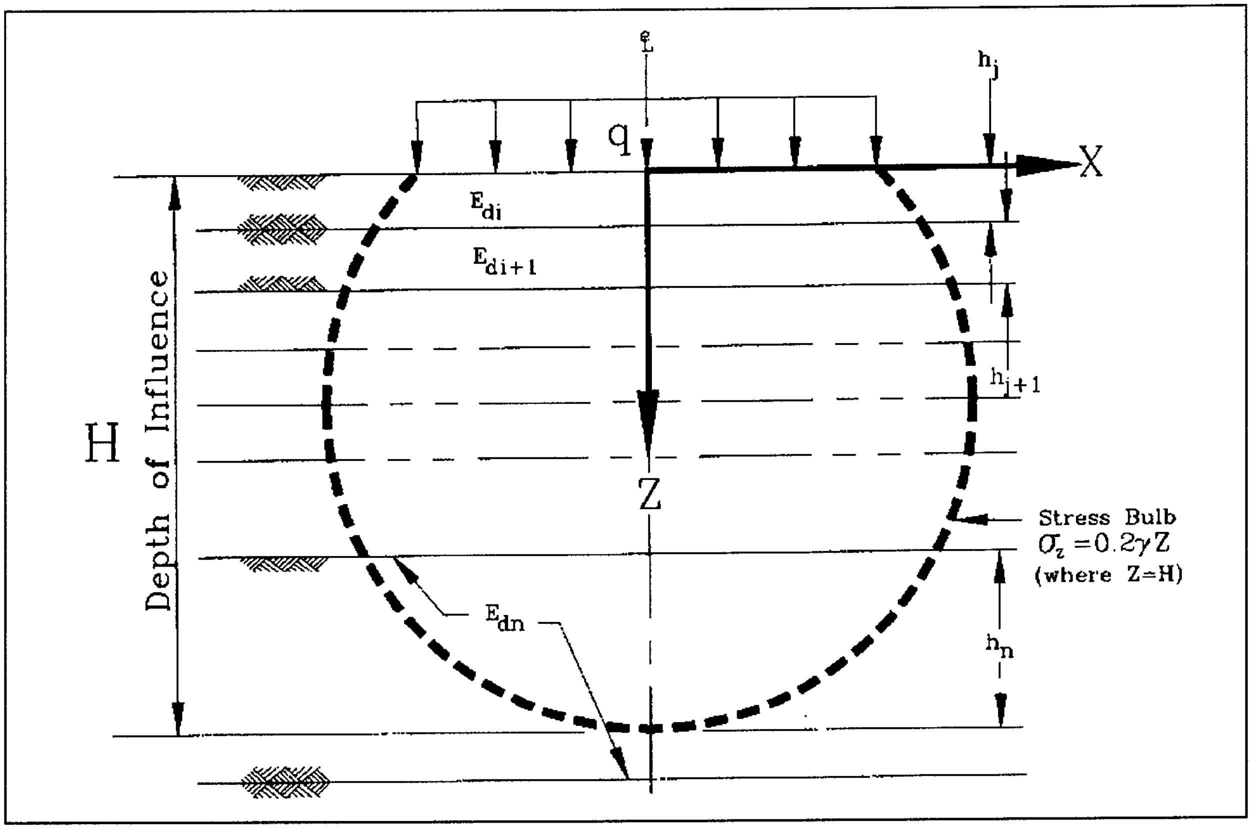


Figure 5-2. Hypothetical foundation underlain by a multilayer strata containing *n* number of layers within the depth of influence

$$E_{dw} = \frac{\sum_{i=1}^{n} \left(E_i \Big/ \sum_{j=1}^{i} h_j \right)}{\sum_{i=1}^{n} \left(1 \Big/ \sum_{j=1}^{i} h_j \right)} \qquad (5\text{-}1)$$

where

E_{dw} = weighted average modulus of deformation

E_{di}, E_{di+1}--E_{dn} = modulus of deformation of each layer. The ratios of any E_{di}, E_{di+1}---E_{dn} terms <3

h_j, h_{j+1}---h_n = thickness of each layer

n = Number of layers

d. Solutions for uniformly loaded rectangular foundations. Rectangular foundations are common shapes for footings and other structures. Solutions for deformation of uniformly loaded foundations are divided into two categories, flexible foundations and rigid foundations.

(1) Flexible foundations. Flexible foundations lack sufficient rigidity to resist flexure under load. As indicated in Figure 5-3 the maximum deformation of a uniformly loaded flexible rectangular foundation occurs at the center of the foundation. The maximum deformation (point *a* in Figure 5-3) can be estimated from the solution of Equation 5-2.

$$\delta_a = \frac{1.12\ qB\ (1 - \mu^2)\ (L/B)^{1/2}}{E_d} \qquad (5\text{-}2)$$

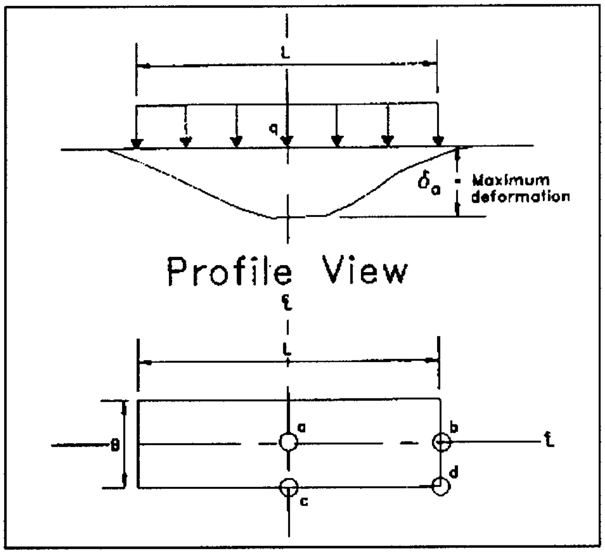


Figure 5-3. Typical deformation profile under a uniformly loaded, rectangular shaped, flexible foundation

where

δ_a = maximum deformation (deformation at point a in Figure 5-3)

q = unit load (force/area)

B = foundation width

L = foundation length

μ = Poisson's ratio of the foundation rock

E_d = modulus of deformation of the foundation rock

Estimates of the deformation of points b, c, and d in Figure 5-3 can be obtained by multiplying the estimated deformation at point a (Equation 5-2) by a reduction factor obtained from Figure 5-4.

(2) Rigid foundations. Rigid foundations are assumed to be sufficiently rigid to resist flexure under load. Examples include concrete gravity structures such as intake and outlet structures. Rigid uniformly loaded foundations settle uniformly. The estimated deformation can be obtained by multiplying the maximum estimated deformation for a flexible foundation of the same dimensions from Equation 5-2 by the reduction factor obtained from the average for rigid load curve in Figure 5-4.

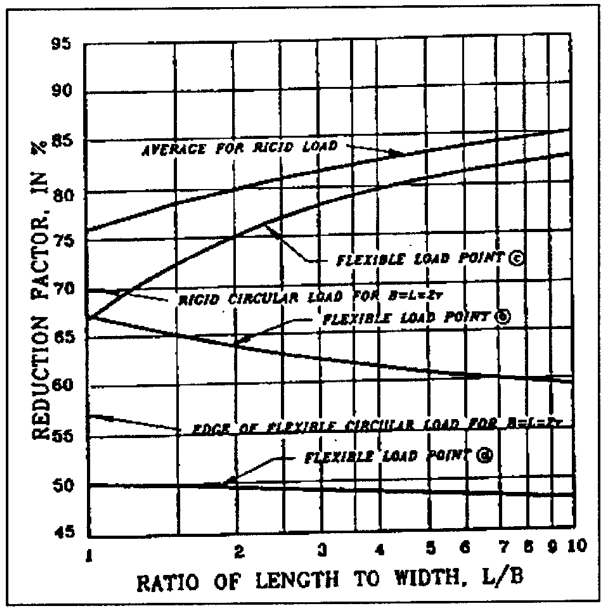


Figure 5-4. Reduction factor in percent of settlement under the center of a flexible rectangular shaped foundation (from NAVDOCKS DM-7)

e. Linearly varying loads. In practice, most gravity retaining structures, such as monoliths of gravity dams and lock walls, do not uniformly distribute loads to the foundation rock. As indicated in Figure 5-5, loading of

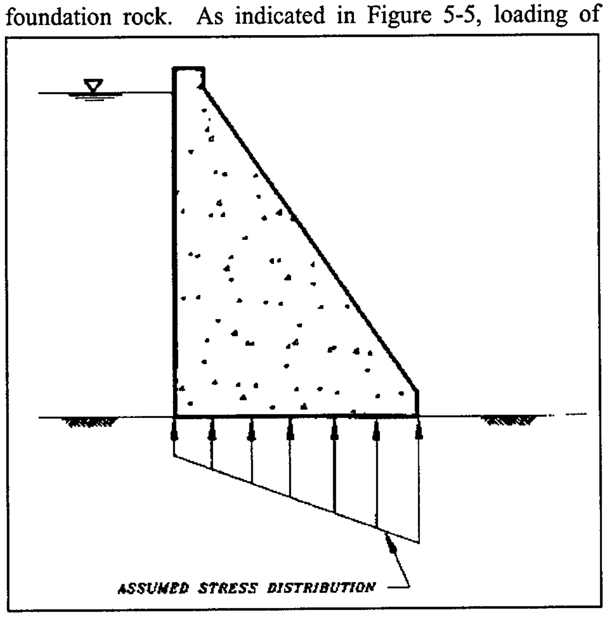


Figure 5-5. Assumed linearly varying stress distribution

these structures may be approximated by assuming linearly varying load distributions. A complete deformation/settlement analyses require the calculation of deformations in both the horizontal and vertical planes. Closed form solutions are available to address linearly varying loads (Poulos and Davis 1974). However, a complete solution requires that the loading conditions be divided into a number of segments. The calculated deformations of each segment are summed to provide a complete solution. In this respect, closed form solutions are tedious, and, because of simplifying assumptions, provide only approximate solutions.

5-7. Numerical Models

Numerical models refer to those analytical methods which, because of their complexity, require the solution of a large number of simultaneous equations. Such solutions are only reasonably possible with the aid of a computer. In many cases numerical models provide the only practical alternative for estimating deformation/settlement of structures subjected to complicated loading conditions and/or are founded on anisotropic, nonhomogeneous rock. Numerical approaches can be separated into two general groups: discontinuum and continuum.

a. Discontinuum models. Discontinuum models feature numerical approaches involving equations of motion for rigid particles or blocks. Such models are frequently referred to as discrete element models. Discontinuum approaches are primarily used when analyzing the stability and/or kinematics of one or more independent and recognizable rock blocks. Because the rock blocks are treated as rigid bodies, discontinuum models are not used to analyze magnitudes of rock deformations.

b. Continuum models. Continuum approaches include the finite element, finite difference, and boundary element methods. All these methods may be used to solve for estimated magnitudes of deformation/settlement. However, the finite element method is the most popular. Numerical modeling of foundation responses dictates the use of constitutive relationships which define material stress-strain behavior. Finite element codes are available which incorporate sophisticated constitutive relationships capable of modeling a variety of nonlinear and/or time-dependent stress-strain behavior. Analytical capabilities offered by some of the more sophisticated codes exceed the ability of the geotechnical engineer to provide meaningful material property parameters. For foundation stress levels and underlying rock types encountered for the majority of structures, reasonable estimates of deformation/settlement can be obtained from linear elastic codes with the modulus of deformation as the primary input parameter. Table 5-1, although not all inclusive, summarizes some of the finite element codes that are commercially available. The choice of code to use should reflect the ability of the code to model the problem at hand and the preference of District office geotechnical professionals charged with the responsibility of settlement analyses.

Section III
Allowable Settlement

5-8. General

For structures founded on rock, the total deformation/settlement seldom controls design. The design for, or control of, differential settlement between critical elements of a structure is essential for the proper and safe functioning of that structure. The total settlement should be computed at a sufficient number of points to establish the overall settlement pattern. From this pattern, the differential settlements can be determined and compared with recommended allowable values.

5-9. Mass Concrete Structure

Mass concrete structures are uniquely designed and constructed to meet the needs of a particular project. These structures vary in size, shape, and intended function between projects. As a result, the magnitude of differential settlement that can be tolerated must be established for each structure. Specifications for the allowable magnitudes of differential deformation/settlement that can be tolerated require the collective efforts of structural and geotechnical professionals, working together as a team. The magnitude of allowable differential movement should be sufficiently low so as to prevent the development of shear and/or tensile stresses within the structure in excess of tolerable limits and to insure the proper functioning of movable features such as lock and flood control gates. For mass concrete structures founded on soft rock, where the modulus of deformation of the rock is significantly less than the elastic modulus of the concrete, there is a tendency for the foundation rock to expand laterally thus producing additional tensile stresses along the base of the foundation. Deere et al. (1967) suggested the following criteria for evaluating the significance of the ratios between the modulus of deformation of the rock (E_{dr}) and the elastic modulus of the concrete (E_c):

Table 5-1
Summary of Finite Element Programs

Program	Capabilities: 2D and 3D Solid Elements	Boundary Elements	Crack Elements	Linear Elastic Anisotropic	Nonlinear Elastic	Plasticity	Viscoelastic or Creep	Interactive Graphics
ABAQUS	X			X	X	X	X	X
ANSYS	X		X	X	X	X	X	X
APPLE-SAP	X	X		X				
ASKA	X		X	X	X	X	X	X
BEASY	X	X				X		
BERSAFE		X	X	X	X	X	X	X
BMINES	X		X	X	X	X	X	X
DIAL	X	X	X	X	X	X	X	X
MCAUTO STRUDL	X			X				X
MSC/ NASTRAN	X		X	X	X	X	X	X
PAFEC	X	X	X	X	X	X	X	X
SAP(WES)	X			X				X
E^3SAP	X			X				X
NONSAP	X		X	X	X	X		X
TITUS	X	X	X	X	X	X		X

a. If E_d/E_c >0.25, then the foundation rock modulus has little effect on stresses generated within the concrete mass.

b. If 0.06 <E_d/E_c <0.25, the foundation rock modulus becomes more significant with respect to stresses generated in the concrete structure. The significance increases with decreasing modulus ratio values.

c. If E_d/E_c <0.06, then the foundation rock modulus almost completely dominates the stresses generated within the concrete. Allowable magnitudes of deformation, in terms of settlement heave, lateral movement, or angular distortion for hydraulic structures should be established by the design team and follow CECW-ED guidance.

Section IV
Treatment Methods

5-10. General

In design cases where the magnitudes of differential deformation/settlement exceed allowable values the team of structural and geotechnical professionals charged with the responsibility of foundation design must make provisions for either reducing the magnitude of differential movement or design the structure to accommodate the differential deformation. A discussion of the latter option is beyond the scope of this manual. There are two approaches available for reducing the magnitude of differential deformation/settlement: improve the rock mass

deformation characteristics and/or modification of the foundation design.

5-11. Rock Mass Improvement

Rock mass improvement techniques refer to techniques which enhances the ability of a rock mass to resist deformation when subjected to an increase in stress. The two techniques that are available include rock reinforcement and consolidation grouting. As a rule, techniques for increasing the modulus of deformation of a rock mass are limited to special cases where only relatively small reductions in deformation are necessary to meet allowable deformation/settlement requirements.

a. Rock reinforcement. Rock reinforcement (i.e. rock bolts, rock anchor, rock tendon, etc.) is primarily used to enhance the stability of structures founded on rock. However, in specialized cases, constraint offered by a systematic pattern of rock reinforcement can be effective in reducing structural movement or translations (for example, rotational deformations of retaining structures). Guidance for rock reinforcement systems is provided in Chapter 9.

b. Consolidation grouting. Consolidation grouting refers to the injection of cementitious grouts into a rock mass for the primary purpose of increasing the modulus of deformation and/or shear strength. The enhancement capabilities of consolidation grouting depend upon rock mass conditions. Consolidation grouting to increase the modulus of deformation is more beneficial in highly fractured rock masses with a predominant number of open joints. Before initiating a consolidation grouting program a pilot field study should be performed to evaluate the potential enhancement. The pilot field study should consist of trial grouting a volume of rock mass representative of the rock mass to be enhanced. In-situ deformation tests (discussed in Chapter 4) should be performed before and after grouting in order to evaluate the degree of enhancement achieved. Guidance pertaining to consolidation grouting is provided in EM 1110-2-3506 and Technical Report REMR-GT-8 (Dickinson 1988).

5-12. Foundation Design Modifications

The most effective means of reducing differential deformation/settlement are through modification of the foundation design. A variety of viable modifications is possible, but all incorporate one or more of three basic concepts: reduce stresses applied to the foundation rock; redistribute the applied stresses to stiffer and more competent rock strata; and in cases involving flexible foundations, reduce maximum deformations by increasing the foundation stiffness. The choice of concept incorporated into the final design depends on the foundation rock conditions, structural considerations, associated cost, and should be accomplished by the design team in accordance with CECW-ED guidance.

Chapter 6
Bearing Capacity

6-1. Scope

This chapter provides guidance for the determination of the ultimate and allowable bearing stress values for foundations on rock. The chapter is subdivided into four sections with the following general topic areas: modes and examples of bearing capacity failures; methods for computing bearing capacity; allowable bearing capacity; and treatment methods for improving bearing capacity.

6-2. Applicability

a. Modes of failure, methods for estimating the ultimate and allowable bearing capacity, and treatments for improving bearing capacity are applicable to structures founded directly on rock or shallow foundations on rock with depths of embedments less than four times the foundation width. Deep foundations such as piles, piers, and caissons are not addressed.

b. As a rule, the final foundation design is controlled by considerations such as deformation/settlement, sliding stability or overturning rather than by bearing capacity. Nevertheless, the exceptions to the rule, as well as prudent design, require that the bearing capacity be evaluated.

Section I
Failure Modes

6-3. General

Bearing capacity failures of structures founded on rock masses are dependent upon joint spacing with respect to foundation width, joint orientation, joint condition (open or closed), and rock type. Figure 6-1 illustrates typical failure modes according to rock mass conditions as modified from suggested modes by Sowers (1979) and Kulhawy and Goodman (1980). Prototype failure modes may actually consist of a combination of modes. For convenience of discussion, failure modes will be described according to four general rock mass conditions: intact, jointed, layered, and fractured.

6-4. Intact Rock Mass

For the purpose of bearing capacity failures, intact rock refers to a rock mass with typical discontinuity spacing (S term in Figure 6-1) greater than four to five times the width (B term in Figure 6-1) of the foundation. As a rule, joints are so widely spaced that joint orientation and condition are of little importance. Two types of failure modes are possible depending on rock type. The two modes are local shear failure and general wedge failure associated with brittle and ductile rock, respectively.

a. Brittle rock. A typical local shear failure is initiated at the edge of the foundation as localized crushing (particularly at edges of rigid foundations) and develops into patterns of wedges and slip surfaces. The slip surfaces do not reach the ground surface, however, ending somewhere in the rock mass. Localized shear failures are generally associated with brittle rock that exhibit significant post-peak strength loss (Figure 6-1a).

b. Ductile rock. General shear failures are also initiated at the foundation edge, but the slip surfaces develop into well defined wedges which extend to the ground surface. General shear failures are typically associated with ductile rocks which demonstrate post-peak strength yield (Figure 6-1b).

6-5. Jointed Rock Mass

Bearing capacity failures in jointed rock masses are dependent on discontinuity spacing, orientation, and condition.

a. Steeply dipping and closely spaced joints. Two types of bearing capacity failure modes are possible for structures founded on rock masses in which the predominant discontinuities are steeply dipping and closely spaced as illustrated in Figure 6-1c and 6-1d. Discontinuities that are open (Figure 6-1c) offer little lateral restraint. Hence, failure is initiated by the compressive failure of individual rock columns. Tightly closed discontinuities (Figure 6-1d) on the other hand, provide lateral restraint. In such cases, general shear is the likely mode of failure.

b. Steeply dipping and widely spaced joints. Bearing capacity failures for rock masses with steeply dipping joints and with joint spacing greater than the width of the foundation (Figure 6-1e) are likely to be initiated by splitting that eventually progresses to the general shear mode.

c. Dipping joints. The failure mode for a rock mass with joints dipping between 20 to 70 degrees with respect to the foundation plane is likely to be general shear (Figure 6-1f). Furthermore, since the discontinuity

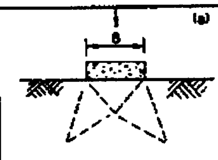
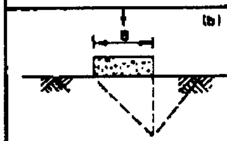
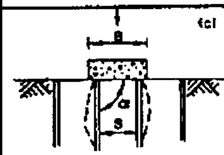
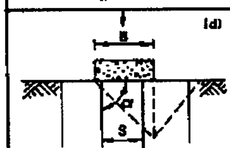
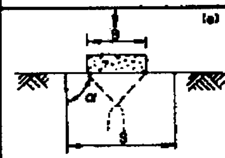
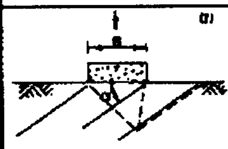
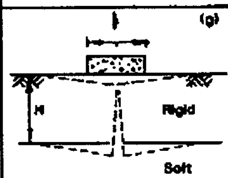
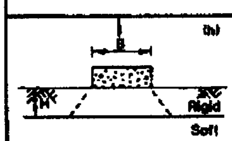
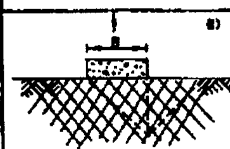
Rock Mass Conditions			Failure		Bearing Capacity Equation No.
	Joint Dip	Joint Spacing	Illustration	Mode	
INTACT	N/A	S>>B	(a) 	Brittle Rock: Local shear failure caused by localized brittle fracture.	Eq. 6.4
			(b) 	Ductile Rock: General shear failure along well defined failure surfaces.	Eq. 6.1
STEEPLY DIPPING JOINTS	70°< α >90°	S<B	(c) 	Open Joints: Compressive failure of individual rock columns. Near vertical joints set(s)	Eq. 6.5
			(d) 	Closed Joints: General shear failure along well defined failure surfaces. Near vertical joint(s).	Eq. 6.1
		S>B	(e) 	Open or Closed Joints: Failure initiated by splitting leading to general shear failure. Near vertical joint set(s).	Eq. 6.6
JOINTED	20°< α <70°	S<B or S>B if failure wedge can develop along joints	(f) 	General shear failure with potential for failure along joints. Moderately dipping joint set(s).	Eq. 6.3
LAYERED	0< α <20	Limiting Values of H with respect to B is dependent upon material properties	(g) 	Thin Rigid Upper Layer: Failure is inlated by tensile failure caused by flexure of thin rigid upper layer.	N/A
			(h) 	Thin Upper Rigid Layer: Failure is initiated by punching tinsile failure of the thin rigid upper layer.	N/A
FRACTURED	N/A	S<<B	(i) 	General shear failure with irregular failure surface through rock mass. Two or more closely spaced joint sets.	Eq. 6.3

Figure 6-1. Typical bearing capacity failure modes associated with various rock mass conditions

represents major planes of weakness, a favorably oriented discontinuity is likely to define at least one surface of the potential shear wedge.

6-6. Layered Rock Mass

Failure modes of multilayered rock masses, with each layer characterized by different material properties, are complicated. Failure modes for two special cases, however, have been identified (Sowers 1979). In both cases the founding layer consists of a rigid rock underlain by a soft highly deformable layer, with bedding planes dipping at less than 20 degrees with respect to the foundation plane. In the first case (Figure 6-1g), a thick rigid layer overlies the soft layer, while in the second case (Figure 6-1h) the rigid layer is thin. In both cases, failure is initiated by tensile failure. However, in the first case, tensile failure is caused by flexure of the rigid thick layer, while in the second case, tensile failure is caused by punching through the thin rigid upper layer. The limiting thickness of the rigid layer in both cases is controlled by the material properties of each layer.

6-7. Highly Fractured Rock Masses

A highly fractured rock mass is one that contains two or more discontinuity sets with typical joint spacings that are small with respect to the foundation width (Figure 6-1i). Highly fractured rock behaves in a manner similar to dense cohesionless sands and gravels. As such, the mode of failure is likely to be general shear.

6-8. Secondary Causes of Failure

In addition to the failure of the foundation rock, aggressive reactions within the rock mineralogy or with ground water or surface water chemistry can lead to bearing capacity failure. Examples include: loss of strength with time typical of some clay shales; reduction of load bearing cross-section caused by chemical reaction between the foundation element and the ground water or surface water; solution-susceptible rock materials; and additional stresses imposed by swelling minerals. Potential secondary causes should be identified during the site investigation phase of the project. Once the potential causes have been identified and addressed, their effects can be minimized.

Section II
Methods for Computing Bearing Capacity

6-9. General

There are a number of techniques available for estimating the bearing capacity of rock foundations. These techniques include analytical methods, traditional bearing capacity equations, and field load tests. Of the various methods, field load tests are the least commonly used for two reasons. First, as discussed in Chapter 4, field load tests, such as the plate bearing test, are expensive. Second, although the test provides information as to the load that will cause failure, there still remains the question of scale effects.

6-10. Definitions

Two terms used in the following discussions require definition. They are the ultimate bearing capacity and allowable bearing value. Definition of the terms are according to the American Society for Testing and Materials.

a. Ultimate bearing capacity. The ultimate bearing capacity is defined as the average load per unit area required to produce failure by rupture of a supporting soil or rock mass.

b. Allowable bearing capacity value. The allowable bearing capacity value is defined as the maximum pressure that can be permitted on a foundation soil (rock mass), giving consideration to all pertinent factors, with adequate safety against rupture of the soil mass (rock mass) or movement of the foundation of such magnitude that the structure is impaired. Allowable bearing values will be discussed in Section III of this chapter.

6-11. Analytical Methods

The ultimate bearing capacity may be implicitly estimated from a number of analytical methods. The more convenient of these methods include the finite element and limit equilibrium methods.

a. Finite element method. The finite element method is particularly suited to analyze foundations with

unusual shapes and/or unusual loading conditions as well as in situations where the foundation rock is highly variable. For example, the potential failure modes for the layered foundation rock cases illustrated in Figures 6-1g and 6-1h will require consideration of the interactions between the soft and rigid rock layers as well as between the rigid rock layer and the foundation. The primary disadvantage of the finite element method is that the method does not provide a direct solution for the ultimate bearing capacity. Such solutions require an analyses of the resulting stress distributions with respect to a suitable failure criterion. In addition to the method's ability to address complex conditions, the primary advantage is that the method provides direct solutions for deformation/ settlement.

b. Limit equilibrium. The limit equilibrium method is applicable to bearing capacity failures defined by general wedge type shear, such as illustrated in Figures 6-1b, 6-1d, 6-1f, and 6-1i. The limit equilibrium method, as applied to sliding stability, is discussed in Chapter 7. Although the principals are the same as in sliding stability solutions, the general form of the equations presented in Chapter 7 needs to be cast in a form compatible with bearing capacity problems. The ultimate bearing capacity corresponds to the foundation loading condition necessary to cause an impending state of failure (i.e. the loading case where the factor of safety is unity).

6-12. Bearing Capacity Equations

A number of bearing capacity equations are reported in the literature which provide explicit solutions for the ultimate bearing capacity. As a rule, the equations represent either empirical or semi-empirical approximations of the ultimate bearing capacity and are dependent on the mode of potential failure as well as, to some extent, material properties. In this respect, selection of an appropriate equation must anticipate likely modes of potential failure. The equations recommended in the following discussions are presented according to potential modes of failure. The appropriate equation number for each mode of failure is given in Figure 6-1.

a. General shear failure. The ultimate bearing capacity for the general shear mode of failure can be estimated from the traditional Buisman-Terzaghi (Terzaghi 1943) bearing capacity expression as defined by Equation 6-1. Equation 6-1 is valid for long continuous foundations with length to width ratios in excess of ten.

$$q_{ult} = cN_c + 0.5\,\gamma BN_\gamma + \gamma DN_q \tag{6-12}$$

where

q_{ult} = the ultimate bearing capacity

γ = effective unit weight (i.e. submerged unit wt. if below water table) of the rock mass

B = width of foundation

D = depth of foundation below ground surface

c = the cohesion intercepts for the rock mass

The terms N_c, N_γ, and N_q are bearing capacity factors given by the following equations.

$$N_c = 2\,N\phi^{1/2}\,(N\phi + 1) \tag{6-2a}$$

$$N\gamma = N\phi^{1/2}\left(N_\phi^2 - 1\right) \tag{6-2b}$$

$$N_q = N_\phi^2 \tag{6-2c}$$

$$N_\phi = \tan^2\,(45 + \phi/2) \tag{6-2d}$$

where

ϕ = angle of internal friction for the rock mass

Equation 6-1 is applicable to failure modes in which both cohesion and frictional shear strength parameters are developed. As such, Equation 6-1 is applicable to failure modes illustrated in Figures 6-1b and 6-1d.

b. General shear failure without cohesion. In cases where the shear failure is likely to develop along planes of discontinuity or through highly fractured rock masses such as illustrated in Figures 6-1f and 6-1i, cohesion cannot be relied upon to provide resistance to failure. In such cases the ultimate bearing capacity can be estimated from the following equation:

$$q_{ult} = 0.5\,\gamma BN_\gamma + \gamma DN_q \tag{6-3}$$

All terms are as previously defined.

c. Local shear failure. Local shear failure represents a special case where failure surfaces start to develop but do not propagate to the surface as illustrated in Figure 6-1a. In this respect, the depth of embedment contributes little to the total bearing capacity stability. An expression for the ultimate bearing capacity applicable to localized shear failure can be written as:

$$q_{ult} = cN_c + 0.5\gamma BN_\gamma \tag{6-4}$$

All terms are as previously defined.

d. Correction factors. Equations 6-1, 6-3, and 6-4 are applicable to long continuous foundations with length to width ratios (L/B) greater than ten. Table 6-1 provides correction factors for circular and square foundations, as well as rectangular foundations with L/B ratios less than ten. The ultimate bearing capacity is estimated from the appropriate equation by multiplying the correction factor by the value of the corresponding bearing capacity factor.

Table 6-1
Correction factors (after Sowers 1979)

Foundation Shape	C_c N_c Correction	$C\gamma$ N_γ Correction
Circular	1.2	0.70
Square	1.25	0.85
Rectangular		
L/B = 2	1.12	0.90
L/B = 5	1.05	0.95
L/B = 10	1.00	1.00

Correction factors for rectangular foundations with L/B ratios other than 2 or 5 can be estimated by linear interpolation.

e. Compressive failure. Figure 6-1c illustrates a case characterized by poorly constrained columns of intact rock. The failure mode in this case is similar to unconfined compression failure. The ultimate bearing capacity may be estimated from Equation 6-5.

$$q_{qult} = 2\ c\ \tan\ (45 + \phi/2) \tag{6-5}$$

All parameters are as previously defined.

f. Splitting failure. For widely spaced and vertically oriented discontinuities, failure generally initiates by splitting beneath the foundation as illustrated in Figure 6-1e. In such cases Bishnoi (1968) suggested the following solutions for the ultimate bearing capacity:

For circular foundations

$$q_{ult} = JcN_{cr} \tag{6-6a}$$

For square foundations

$$q = 0.85JcN_{cr} \tag{6-6b}$$

For continuous strip foundations for L/B ≤ 32

$$q_{ult} = JcN_{cr}/(2.2 + 0.18\ L/B) \tag{6-6c}$$

where

J = correction factor dependent upon thickness of the foundation rock and width of foundation.

L = length of the foundation

The bearing capacity factor N_{cr} is given by:

$$N_{cr} = \frac{2N_\phi^2}{1+N_\phi}\ (\cot\phi)\ (S/B)\ \left(1 - \frac{1}{N_\phi}\right) - N\phi\ (\cot\phi) + 2N\phi^{1/2} \tag{6-6d}$$

All other terms are as previously defined. Graphical solutions for the correction factor (J) and the bearing capacity factor (N_{cr}) are provided in Figures 6-2 and 6-3, respectively.

g. Input parameters. The bearing capacity equations discussed above were developed from considerations of the Mohr-Columb failure criteria. In this respect, material property input parameters are limited to two parameters; the cohesion intercept (c) and the angle of internal friction (ϕ). Guidance for selecting design shear strength parameters is provided in Chapter 4. However, since rock masses generally provide generous margins of safety against bearing capacity failure, it is recommended that

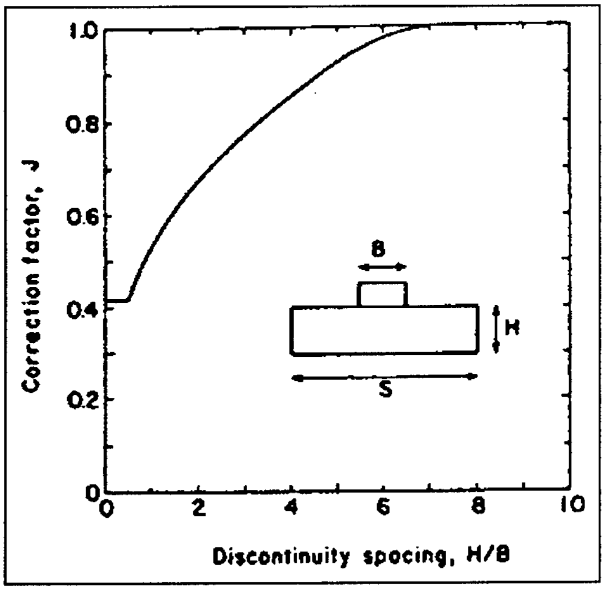


Figure 6-2. Correction factor for discontinuity spacing with depth (after Bishnoi 1968)

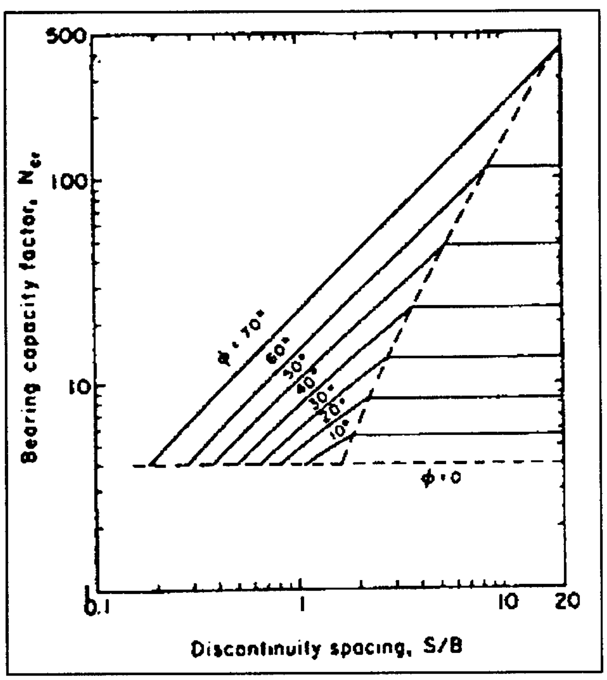


Figure 6-3. Bearing capacity factor for discontinuity spacing (after Bishnoi 1968)

initial values of c and ϕ selected for assessing bearing capacity be based on lower bound estimates. While inexpensive techniques are available on which to base lower bound estimates of the friction angle, no inexpensive techniques are available for estimating lower bound cohesion values applicable to rock masses. Therefore, for computing the ultimate bearing capacity of a rock mass, the lower bound value of cohesion may be estimated from the following equation.

$$c = \frac{q_u \quad (s)}{2 \tan\left(45 + \frac{\phi}{2}\right)} \tag{6-7a}$$

where

q_u = unconfined compressive strength of the intact rock from laboratory tests.

$$s = \exp \frac{(RMR - 100)}{9} \tag{6-7b}$$

All other parameters are as previously defined.

6-13. Eccentric Load on a Horizontal Foundation

Eccentric loads acting on foundations effectively reduce the bearing capacity. Figure 6-4a illustrates a typical structure subjected to an eccentric load. In order to prevent loss of rock/structure contact at the minimum stress edge of the foundation (Figure 6-4a), the structure must be designed so that the resultant of all forces acting on the foundations passes through the center one-third of the foundation. As indicated in Figure 6-4a, the stress distribution can be approximated by linear relationship. Equations 6-8a and 6-8b define the approximate maximum and minimum stress, respectively.

$$q_{(max)} = \frac{Q}{B}\left(1 + \frac{6e}{B}\right) \tag{6-8a}$$

$$q_{(min)} = \frac{Q}{B}\left(1 - \frac{6e}{B}\right) \tag{6-8b}$$

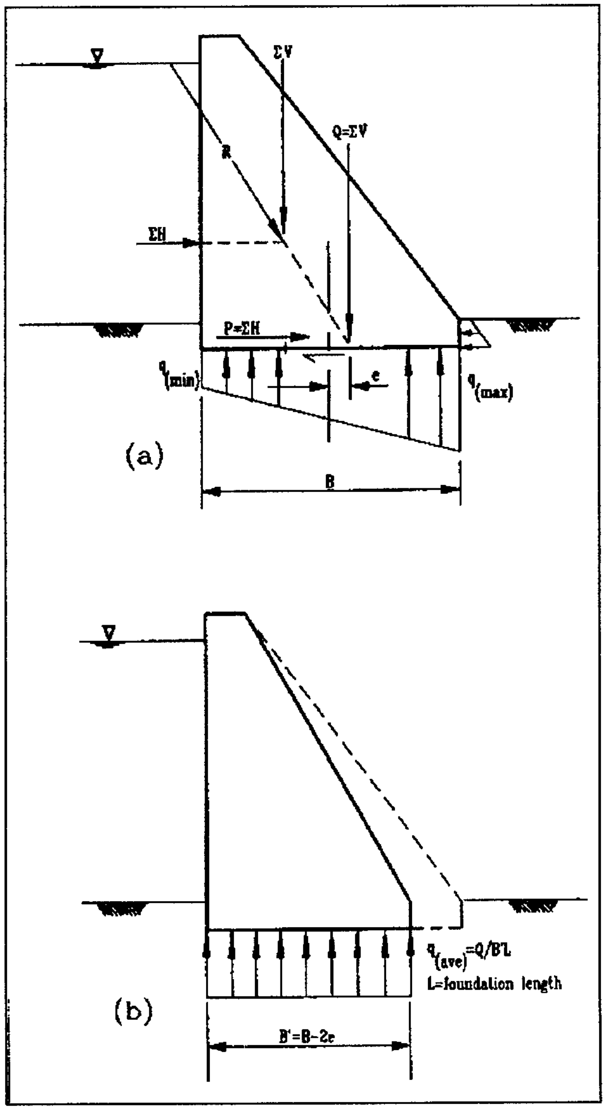


Figure 6-4. Typical eccentrically loaded structure foundation

where

$q_{(max)}$ = maximum stress

$q_{(min)}$ = minimum stress

Q = vertical force component of the resultant of all forces acting on the structure

B = the foundation width

e = distance from the center of the foundation to the vertical force component Q

The ultimate bearing capacity of the foundation can be approximated by assuming that the vertical force component Q is uniformly distributed across a reduced effective foundation width as indicated in Figure 6-4b. The effective width is defined by the following equation.

$$B' = B - 2e \tag{6-10}$$

The effective width (B') is used in the appropriate bearing capacity equation to calculate the ultimate bearing capacity.

6-14. Special Design Cases

The bearing capacity equations discussed above are applicable to uniformly loaded foundations situated on planar surfaces. Frequently, designs suited to the particular requirements of a project require special considerations. Special design cases for which solutions of the ultimate bearing capacity are readily available are summarized in Figure 6-5. As indicated in Figure 6-5, these special cases include inclined loads, inclined foundations, and foundations along or near slopes. Guidance for these special cases is provided in EM 1110-2-2502 and the NAVDOCKS DM-7. Ultimate bearing capacity solutions for special design cases should be in keeping with the modes of failure summarized in Figure 6-1.

Section III
Allowable Bearing Capacity Value

6-15. General

The allowable bearing capacity value is defined in paragraph 6-10*b*. In essence, the allowable bearing capacity is the maximum limit of bearing stress that is allowed to be applied to the foundation rock. This limiting value is intended to provide a sufficient margin of safety with respect to bearing failures and deformation/settlement. Nevertheless, a prudent design dictates that, once the allowable bearing capacity value has been determined, a separate calculation be performed in order to verify that

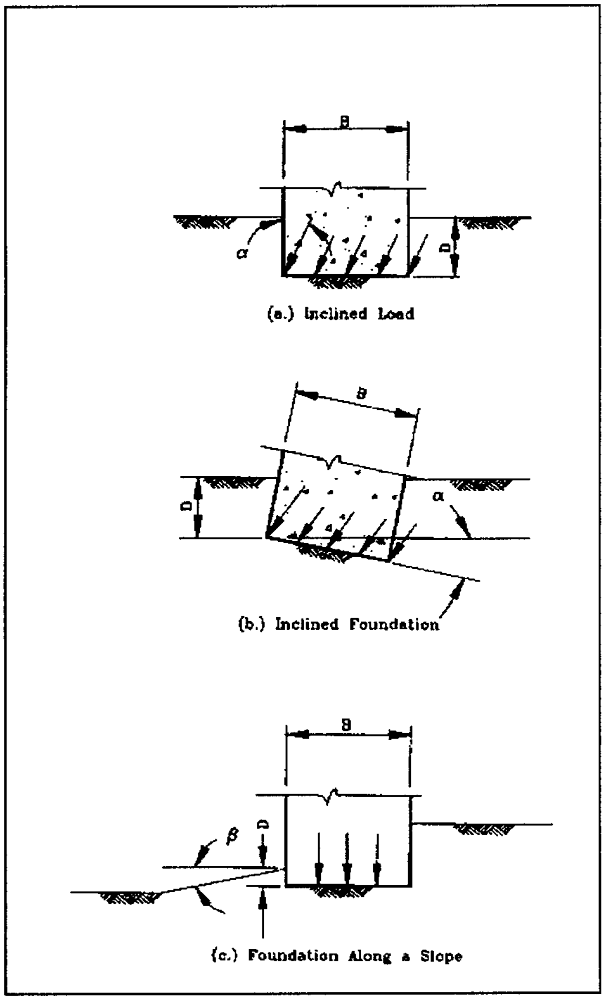


Figure 6-5. Special foundation design cases

the allowable differential deformation/settlement is not exceeded.

6-16. Determination

There are at least three approaches for determining allowable bearing capacity values. First, the allowable value may be determined by applying a suitable factor of safety to the calculated ultimate bearing capacity. The selection of final allowable bearing values used in design of hydraulic structures must be based on the factor of safety approach in which all site specific conditions and unique problems of such structures are considered. Second, allowable values may be obtained from various building codes. However, building codes, in general, apply only to residential or commercial buildings and are not applicable to the unique problems of hydraulic structures. Finally, allowable values may be obtained from empirical correlations. As a rule, empirical correlations are not site specific and hence should be used only for preliminary design and/or site evaluation purposes. Regardless of the approach used, the allowable value selected for final design must not exceed the value obtained from the factor of safety considerations discussed in paragraph 6-16*a*.

a. Factor of safety. The allowable bearing capacity value, q_a, based on the strength of the rock mass is defined as the ultimate bearing capacity, q_{ult}, divided by a factor of safety (*FS*):

$$q_a = q_{ult}/FS \tag{6-11}$$

The average stress acting on the foundation material must be equal to or less than the allowable bearing capacity according to the following equation.

$$Q/BL \le q_a \tag{6-12}$$

For eccentrically loaded foundations the B' value (i.e. Equation 6-10) is substituted for the B term in Equation 6-12. The factor of safety considers the variability of the structural loads applied to the rock mass, the reliability with which foundation conditions have been determined, and the variability of the potential failure mode. For bearing capacity problems of a rock mass, the latter two considerations are the controlling factors. For most structural foundations, the minimum acceptable factor of safety is 3 with a structural load comprised of the full dead load plus the full live load.

b. Building codes. Allowable bearing capacity values that consider both strength and deformation/settlement are prescribed in local and national building codes. Local codes are likely to include experience and geology within their jurisdiction while national codes are more generic. For example, a local code will likely specify a particular rock formation such as "well-cemented Dakota sandstone" while a national code may use general terminology such as "sedimentary rock in sound condition." As a rule, allowable values recommended by the building codes are conservative.

c. Empirical correlations. Peck, Hanson, and Thornburn (1974) suggested an empirical correlation between the allowable bearing capacity stress and the RQD, as shown in Figure 6-6. The correlation is intended

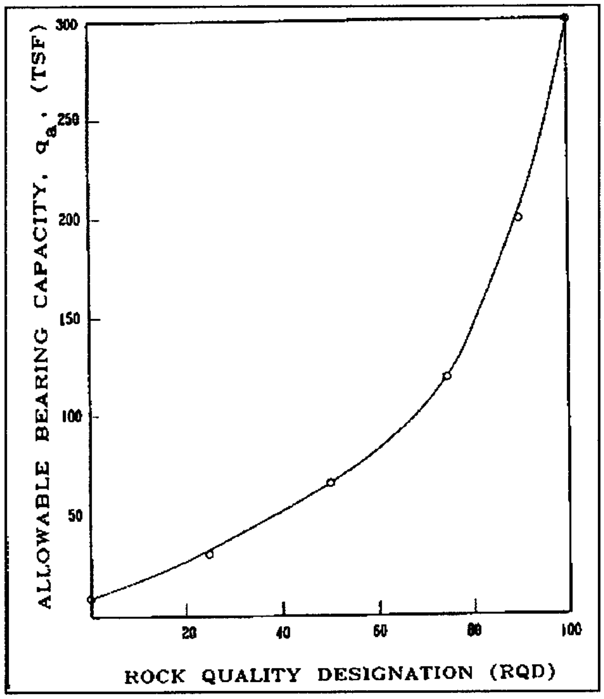


Figure 6-6. Allowable contact pressure on jointed rock

for a rock mass with discontinuities that "are tight or are not open wider than a fraction of an inch."

6-17. Structural Limitations

The maximum load that can be applied to a rock foundation is limited by either the rock's ability to sustain the force without failure or excessive settlement, or the ability of the substructure to sustain the load without failure or excessive deformation. In some cases the structural design of the foundation element will dictate the minimum element size, and, consequently, the maximum contact stress on the rock. For typical concrete strengths in use today, the strength of the concrete member is significantly less than the bearing capacity of many rock masses.

Section IV
Treatment Methods

6-18. General

Treatment methods for satisfying bearing capacity requirements are essentially the same as those for satisfying deformation/settlement requirements discussed in Chapter 5. In addition to the previously discussed methods, an examination of the general ultimate bearing capacity equation (i.e. Equation 6-1) indicates the importance of two parameters not directly related to deformability. These two parameters are the effective unit weight of the foundation rock and the depth of the foundation below the ground surface.

6-19. Effective Unit Weight

For foundations below the water table the effective unit weight is the unit weight of the foundation rock minus the unit weight of water (i.e. submerged unit weight of the rock). Hence, foundations located above the water table will develop significantly more resistance to potential bearing capacity failures than foundations below the water table.

6-20. Foundation Depth

Foundations constructed at greater depths may increase the ultimate bearing capacity of the foundation. The improved capacity is due to a greater passive resisting force and a general increase in rock mass strength with depth. The increased lithostatic pressure closes discontinuities, and the rock mass is less susceptible to surficial weathering. Occasionally, deeper burial may not be advantageous. A region with layers of differing rock types may contain weaker rock at depth. In such an instance, a strong rock might overlie a layer such as mudstone, or, if in a volcanic geology, it might be underlain by a tuff or ash layer. In these instances, deeper burial may even decrease the bearing capacity. The geologic investigation will determine this possibility.

Chapter 7
Sliding Stability

7-1. Scope

This chapter provides guidance for assessing the sliding stability of laterally loaded structures founded on rock masses. Examples of applicable structures include gravity dams, coffer dams, flood walls, lock walls, and retaining structures. The chapter is divided into three sections to include: modes of failure; methods of analyses; and treatment methods.

Section I
Modes of Failure

7-2. General

Paths along which sliding can occur will be confined to the foundation strata; pass through both the foundation strata and the structure; or just pass through the structure. This chapter addresses sliding where the failure path is confined to the foundation strata or at the interface between the strata and the structure's foundation. Although complex, foundation-structure sliding failure or sliding failure through the structure are conceptually possible and must be checked, such failures are likely to occur only in earth structures (e.g., embankments). The analyses of these later two failure modes are addressed in EM 1110-2-1902.

7-3. Potential Failure Paths

Potential failure paths along which sliding may occur can be divided into five general categories as illustrated in Figure 7-1.

a. Failure along discontinuities. Figure 7-1a illustrates a mode of potential failure where the failure path occurs along an unfavorably oriented discontinuity. The mode of failure is kinematically possible in cases where one or more predominate joint sets strike roughly parallel to the structure and dip in the upstream direction. The case is particularly hazardous with the presence of an additional joint set striking parallel to the structure and dipping downstream. In the absence of the additional joint set, failure is generally initiated by a tensile failure at the heel of the structure. Where possible the structure should be aligned in a manner that will minimize the development of this potential mode of failure.

b. Combined failure. A combined mode of failure is characterized by situations where the failure path can occur both along discontinuities and through intact rock as illustrated in Figure 7-1b. Conceptually, there are any number of possible joint orientations that might result in a combined mode of failure. However, the mode of failure is more likely to occur in geology where the rock is horizontally or near horizontally bedded and the intact rock is weak.

c. Failure along interface. In cases where structures are founded on rock masses containing widely spaced discontinuities, none of which are unfavorably oriented, the potential failure path is likely to coincide with the interface between the structure and the foundation strata. The interface mode of failure is illustrated in Figure 7-1c.

d. Generalized rock mass failure. In the generalized rock mass mode of failure, the failure path is a localized zone of fractured and crushed rock rather than well defined surfaces of discontinuity. As implied in Figure 7-1d, a generalized rock mass failure is more likely to occur in highly fractured rock masses.

e. Buckling failure. Figure 7-1e illustrates a conceptual case where failure is initiated by buckling of the upper layer of rock downstream of the structure. Rock masses conducive to buckling type failures would contain thin, horizontally bedded, rock in which the parent rock is strong and brittle. Although no case histories have been recorded where buckling contributed to or caused failure, the potential for a buckling failure should be addressed where warranted by site conditions.

Section II
Methods of Analysis

7-4. General Approach

The guidance in this chapter is based on conventional geotechnical principles of limit equilibrium. The basic principle of this method applies the factor of safety to the least known conditions affecting sliding stability, this is, the material shear strength. Mathematically, the basic principle is expressed as:

$$\tau = \frac{\tau_f}{FS} \tag{7-1}$$

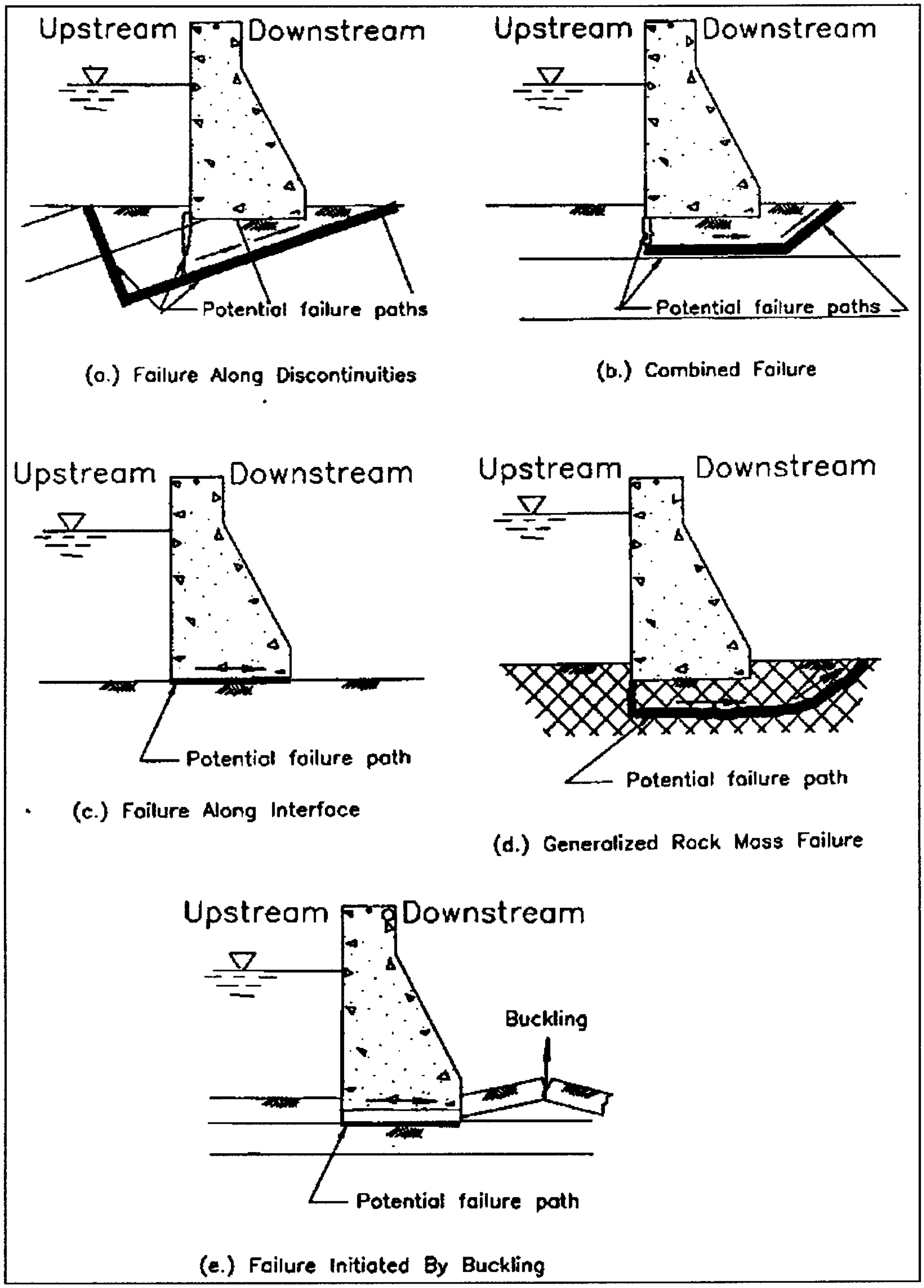


Figure 7-1. Potential failure paths

in which τ is the limiting (applied) shear stress required for equilibrium and τ_f is the maximum available shear strength that can be developed. The ratio of these two quantities, expressed by Equation 7-2, is called the factor of safety.

$$FS = \tau_f / \tau \tag{7-2}$$

The maximum available shear strength τ_f is defined by the Mohr-Coulomb failure criterion. Procedures for selecting the appropriate shear strength parameters c and ϕ are discussed in Chapter 4.

7-5. Conditions for Stability

According to this method, the foundation is stable with respect to sliding when, for any potential slip surface, the resultant of the applied shear stresses required for equilibrium is smaller than the maximum shear strength that can be developed. A factor of safety approaching unity for any given potential slip surface implies failure by sliding is impending. The surface along which sliding has the greatest probability of occurring is the surface that results in the smallest factor of safety. This surface is referred to as the potential critical failure surface.

7-6. Assumptions

As in any mathematical expression which attempts to model a geologic phenomenon, the limit equilibrium method requires the imposition of certain simplifying assumptions. Assumptions invariably translate into limitations in application. Limit equilibrium methods will provide an adequate assessment of sliding stability provided that sound engineering judgment is exercised. This judgment requires a fundamental appreciation of the assumptions involved and the resulting limitations imposed. The following discussion emphasizes the more important assumptions and limitations.

a. Failure criterion. Conventional limit equilibrium solutions for assessing sliding stability incorporate the linear Mohr-Coulomb failure criterion (see Figure 4-5) for estimating the maximum available shear strength (τ_f). It is generally recognized that failure envelopes for all modes of rock failure are, as a rule, non-linear. As discussed in Chapter 4, imposition of a linear criterion for failure, as applied to rock, requires experience and judgment in selecting appropriate shear strength parameters.

b. Two-dimensional analysis. The method presented in this chapter is two-dimensional in nature. In most cases, problems associated with sliding in rock masses involve the slippage of three-dimensional wedges isolated by two or more discontinuities and the ground surface. In such cases, a two-dimensional analysis generally results in a conservative assessment of sliding stability. It is possible for a two-dimensional analysis to predict an impending failure where in reality the assumed failure mechanism is kinematically impossible.

c. Failure surface. The stability equations are based on an assumed failure surface consisting of one or more planes. Multiplane surfaces form a series of wedges which are assumed to be rigid. The analysis follows the method of slices approach common to limit equilibrium generalized slip surfaces used in slope stability analysis (e.g., see Janbu 1973). Slices are taken at the intersection of potential failure surface planes. Two restrictions are imposed by the failure surface assumptions. First, the potential failure surface underlying the foundation element is restricted to one plane. Second, planear surfaces are not conducive to search routines to determine the critical potential failure surface. As a result, determination of the critical failure surface may require a large number of trial solutions; particularly in rock masses with multiple, closely spaced, joint sets.

d. Force equilibrium. Equations for assessing stability were developed by resolving applied and available resisting stresses into forces. The following assumptions are made with respect to forces.

(1) Only force equilibrium is satisfied. Moment equilibrium is not considered. Stability with respect to overturning must be determined separately.

(2) In order to simplify the stability equations, forces acting vertically between wedges are assumed to be zero. Neglecting these forces generally results in a conservative assessment of sliding stability.

(3) Because only forces are considered, the effects of stress concentrations are unknown. Potential problems associated with stress concentrations must be addressed separately. The finite element method is ideally suited for this task.

e. Strain compatibility. Considerations regarding displacements are excluded from the limit equilibrium approach. The relative magnitudes of the strain at failure for different foundation materials may influence the results of the sliding stability analysis. Such complex structure-foundation systems may require a more intensive sliding investigation than a limit equilibrium approach. In

this respect, the effects of strain compatibility may require special interpretation of data from in-situ tests, laboratory tests, and finite element analyses.

f. Factor of safety. Limit equilibrium solutions for sliding stability assume that the factor of safety of all wedges are equal.

7-7. Analytical Techniques for Multi-Wedge Systems

a. General wedge equations. The general wedge equations are derived from force equilibrium of all wedges in a system of wedges defined by the geometry of the structure and potential failure surfaces. Consider the ith wedge in a system of wedges illustrated in Figure 7-2. The necessary geometry notation for the ith wedge and adjacent wedges are as shown (Figure 7-2). The origin of the coordinate system for the wedge considered is located in the lower left hand corner of the wedge. The x and y axes are horizontal and vertical respectively. Axes which are tangent (t) and normal (n) to the failure plane are oriented at an angle (α) with respect to the $+x$ and $+y$ axes. A positive value of α is a counterclockwise rotation, a negative value of α is a clockwise rotation. The distribution of pressures/stresses with resulting forces is illustrated in Figure 7-3. Figure 7-4 illustrates the free body diagram of the resulting forces. Summing the forces normal and tangent to the potential failure surface and solving for $(P_{i-1} - P_i)$ results in the following equation for the ith wedge:

$$\begin{aligned}(P_{i-1} - P_i) = \Big[&((W_i + V_i)\cos\alpha_i \\ &- U_i + (H_{Li} - H_{Ri})\sin\alpha_i) \\ &\frac{\tan\phi_i}{FS_i} - (H_{Li} - H_{Ri})\cos\alpha_i \\ &+ (W_i + V_i)\sin_{\alpha_i} + \frac{C_i}{FS_i}L_i\Big] \\ &\div \left[\cos\alpha_i - \sin\alpha_i\,\frac{\tan\phi_i}{FS_i}\right]\end{aligned} \tag{7-3}$$

where

i = subscript notation for the wedge considered

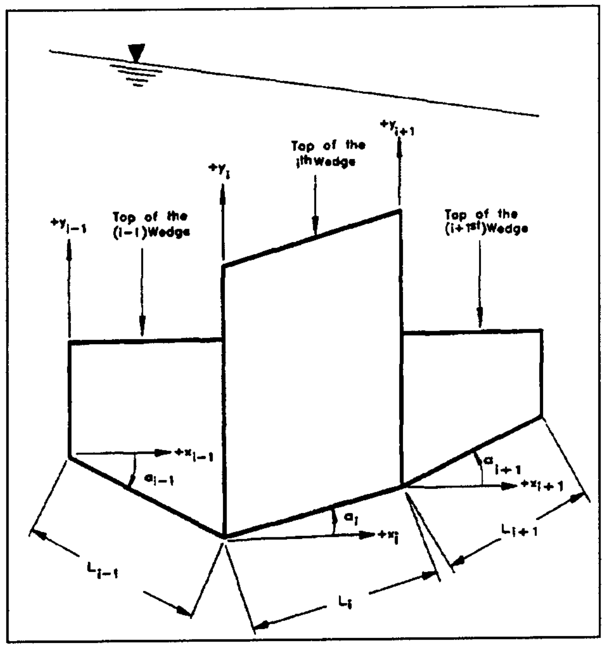


Figure 7-2. Hypothetical i[th] wedge and adjacent wedges subject to potential sliding

P = horizontal residual forces acting between wedges as a result of potential sliding

W = the total weight of wedge to include rock, soil, concrete and water (do not use submerged weights)

V = any vertical force applied to the wedge

α = angle of potential failure plane with respect to the horizontal ($-\alpha$ denotes downslope sliding, $+\alpha$ denotes upslope sliding)

U = the uplift force exerted on the wedge at the potential failure surface

H = in general, any horizontal force applied to the wedge (H_L and H_R refers to left and right hard forces as indicated in Figures 7-3 and 7-4)

L = the length of the wedge along the potential failure surface

FS = the factor of safety

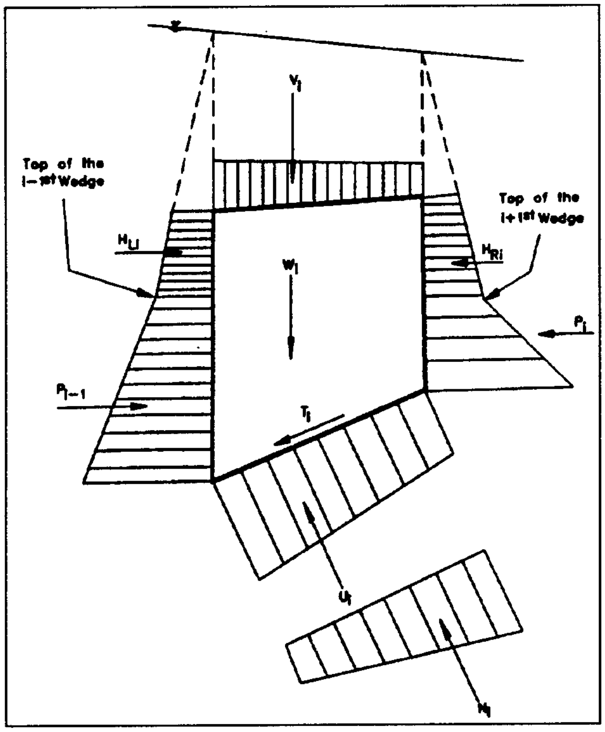


Figure 7-3. Distribution of pressures, stresses and resultant forces acting on the hypothetical i[th] wedge

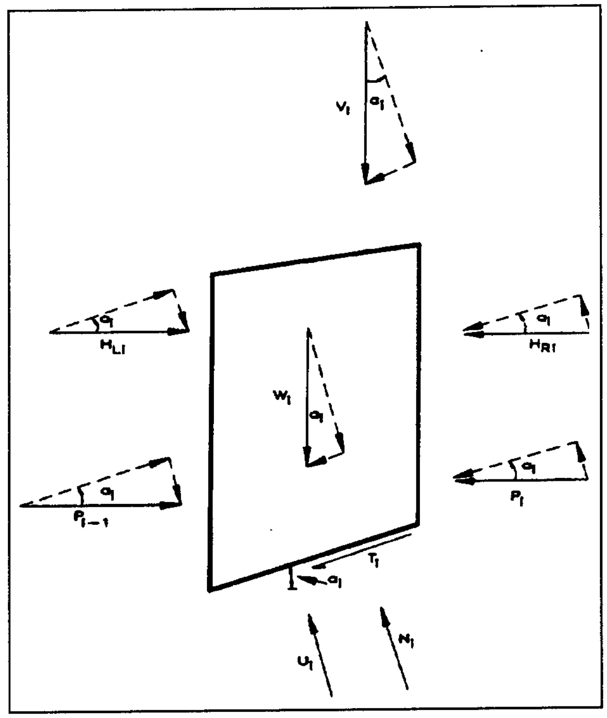


Figure 7-4. Free body diagram of the hypothetical i[th] wedge

c = the cohesion shear strength parameter

ϕ = the angle of internal friction

b. Equilibrium requirements. An inspection of Equation 7-3 reveals that for a given wedge there will be two unknowns (i.e., $(P_{i-1} - P_i)$ and FS). In a wedge system with n number of wedges, Equation 7-3 will provide n number of equations. Because FS is the same for all wedges there will be $n + 1$ unknowns with n number of equations for solution. The solution for the factor of safety is made possible by a conditional equation establishing horizontal equilibrium of the wedge system. This equation states that the sum of the differences in horizontal residual forces $(P_{i-1} - P_i)$ acting between wedges must equal the differences in the horizontal boundary forces. Since boundary forces are usually equal to zero, the conditional equation is expressed as

$$\sum_{i=1}^{i=n} (P_{i-1} - P_i) = 0 \qquad (7\text{-}4)$$

where

n = the total number of wedges in the system.

c. Alternate equation. An alternate equation for the implicit solution of the factor of safety for a system of n wedges is given below:

$$FS = \frac{\displaystyle\sum_{i=1}^{i=n} \frac{C_i L_i \cos\alpha_i + (W_i + V_i - U_i \cos\alpha_i)\tan\phi_i}{n_{\alpha_i}}}{\displaystyle\sum_{i=1}^{i=n} [H_i - (W_i + V_i)\tan\alpha_i]} \qquad (7\text{-}5a)$$

where

$$n_{\alpha_i} = \frac{1 - \dfrac{\tan\phi_i \tan\alpha_i}{FS}}{1 + \tan^2\alpha_i} \qquad (7\text{-}5b)$$

All other terms are as defined above. The derivation of Equations 7-5 follows that of Equations 7-3 and 7-4 except that forces are summed with respect to the x and y coordinates.

7-8. Preliminary Procedures

Factor of safety solutions for a multi-wedge system containing a number of potential failure surfaces can result in a significant book-keeping problem. For this reason, it is recommended that prior to the analytical solution for the factor of safety, the following preliminary procedures be implemented.

a. Define and identify on a scale drawing all potential failure surfaces based on the stratification, location, orientation, frequency, and distribution of discontinuities within the foundation material as well as the geometry, location, and orientation of the structure.

b. For each potential failure surface, divide the mass into a number of wedges. A wedge must be created each time there is a change in slip plane orientation and/or a change in shear strength properties. However, there can be only one structural wedge.

c. For each wedge draw a free body diagram which shows all the applied and resulting forces acting on that wedge. Include all necessary dimensions on the free body diagram. Label all forces and dimensions according to the appropriate parameter notations discussed above.

d. Prepare a table, which lists all parameters, to include shear strength parameters for each wedge in the system of wedges defining the potential slip mass.

7-9. Analytical Procedures

While both the general wedge equation and the alternate equation will result in the same calculated factor of safety for a given design case, the procedure for calculating that value is slightly different. Solutions for hypothetical example problems are provided in EM 1110-2-2200 and Nicholson (1983a).

a. General wedge method. The solution for the factor of safety using Equations 7-3 and 7-4 requires a trial-and-error procedure. A trial value for the factor of safety, *FS*, is inserted in Equation 7-3 for each wedge to obtain values of the differences in horizontal residual *P* forces acting between wedges. The differences in *P* forces for each wedge are then summed; a negative value indicates that the trial value of *FS* was to high and conversely a positive value indicates that the trial value of *FS* was too low. The process is repeated until the trial *FS* value results in an equality from Equation 7-4. The value of *FS* which results in an equality is the correct value for the factor of safety. The number of trial-and-error cycles can be reduced if trial values of *FS* are plotted with respect to the sum of the differences of the *P* forces (see examples in EM 1110-2-2200 and Nicholson (1983a)).

b. Alternate methods. Equations 7-5a and 7-5b, when expanded, can be used to solve for the factor of safety for a system containing one or more wedges. Since the n_α term, defined by Equation 7-5b, is a function of *FS*, the solution for *FS* requires an iterative process. An assumed initial value of *FS* is inserted into the n_α term for each wedge in the expanded form of Equation 7-5a, and a new factor of safety is calculated. The calculated factor of safety is then inserted into the n_α term. The process is repeated until the inserted value of *FS* equals the calculated value of *FS*. Convergence to within two decimal places usually occurs in 3 to 4 iteration cycles.

c. Comparison of methods. The general wedge equation (Equation 7-3) was formulated in terms of the difference in horizontal boundary forces to allow the design engineer to solve directly for forces acting on the structure for various selected factors of safety. The procedure has an advantage for new structures in that it allows a rapid assessment of the horizontal forces necessary for equilibrium for prescribed factors of safety. The alternate equation (Equation 7-5a and 7-5b) solves directly for *FS*. Its advantage is in the assessment of stability for existing structures. Both equations are mathematically identical (Nicholson 1983a).

7-10. Design Considerations

Some special considerations for applying the general wedge equation to specific site conditions are discussed below.

a. Active wedge. The interface between the group of active wedges and the structural wedge is assumed to be a vertical plane located at the heel of the structural wedge and extending to the base of the structural wedge. The magnitudes of the active forces depend on the actual values of the safety factor, the inclination angles (α) of the slip path, and the magnitude of the shear strength that can be developed. The inclination angles, corresponding to the maximum active residual *P* forces for each potential failure surface, can be determined by independently analyzing the group of active wedges for trial safety factors. In rock the inclination may be predetermined by discontinuities in the foundation.

b. Structural wedge. Discontinuities in the slip path beneath the structural wedge should be modeled by

assuming an average slip-plane along the base of the structural wedge.

c. Passive wedge. The interface between the group of passive wedges and the structural wedge is assumed to be a vertical plane located at the toe of the structural wedge and extending to the base of the structural wedge. The magnitudes of the passive residual *P* forces depend on the actual values of the safety factor, the inclination angles of the slip path, and the magnitude of shear strength that can be developed. The inclination angles, corresponding to the minimum passive residual *P* forces for each potential failure mechanism, can be estimated by independently analyzing the group of passive wedges for trial safety factors. When passive resistance is used special considerations must be made. Removal of the passive wedge by future construction must be prevented. Rock that may be subjected to high velocity water scouring should not be used unless amply protected. Also, the compressive strength of the rock layers must be sufficient to develop the wedge resistance. In some cases wedge resistance should not be assumed without resorting to special treatment such as installing rock anchors.

d. Tension cracks. Sliding analyses should consider the effects of cracks on the active side of the structural wedge in the foundation material due to differential settlement, shrinkage, or joints in a rock mass. The depth of cracking in cohesive foundation material can be estimated in accordance with the following equations.

$$d_c = \frac{2c_d}{\gamma} \tan\left(45 - \frac{\phi_d}{2}\right) \tag{7-6a}$$

where

$$c_d = \frac{c}{FS} \tag{7-6b}$$

$$\phi_d = \tan^{-1}\left(\frac{\tan\phi}{\mathrm{FS}}\right) \tag{7-6c}$$

The value (d_c) in a cohesive foundation cannot exceed the embedment of the structural wedge. The depth of cracking in massive, strong, rock foundations should be assumed to extend to the base of the structural wedge. Shearing resistance along the crack should be ignored and full hydrostatic pressure should be assumed to extend to the bottom of the crack. The hydraulic gradient across the base of the structural wedge should reflect the presence of a crack at the heel of the structural wedge.

e. Uplift without drains. The effects of seepage forces should be included in the sliding analysis. Analyses should be based on conservative estimates of uplift pressures. Estimates of uplift pressures on the wedges can be based on the following assumptions:

(1) The uplift pressure acts over the entire area of the base.

(2) If seepage from headwater to tailwater can occur across a structure, the pressure head at any point should reflect the head loss due to water flowing through a medium. The approximate pressure head at any point can be determined by the line-of-seepage method. This method assumes that the head loss is directly proportional to the length of the seepage path. The seepage path for the structural wedge extends from the upper surface (or internal ground-water level) of the uncracked material adjacent to the heel of the structure, along the embedded perimeter of the structural wedge, to the upper surface (or internal ground-water level) adjacentto the toe of the structure. Referring to Figure 7-5, the seepage distance is defined by points a, b, c, and d. The pressure head at any point is equal to the elevation head minus the product of the hydraulic gradient times the distance along the seepage path to the point in question. Estimates of pressure heads for the active and passive wedges should be consistent with those of the heel and toe of the structural wedge.

(3) For a more detailed discussion of the line-of-seepage method, refer to EM 1110-2-2502, Retaining and Flood Walls. For the majority of structural stability computations, the line-of-seepage is considered sufficiently accurate. However, there may be special situations where the flow net method is required to evaluate seepage problems.

f. Uplift with drains. Uplift pressures on the base of the structural wedge can be reduced by foundation drains. The pressure heads beneath the structural wedge developed from the line-of-seepage analysis should be modified to reflect the effects of the foundation drains. The maximum pressure head along the line of foundation drains can be estimated from Equation 7-7:

$$U_x = U_1 + R\left(\frac{L - x}{L}\right)(U_2 - U_1) \tag{7-7}$$

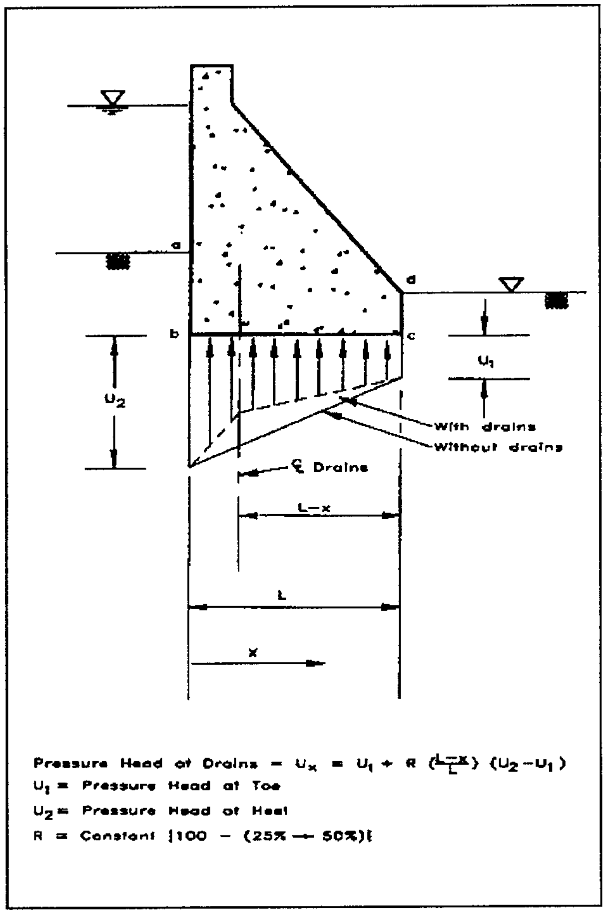


Figure 7-5. Uplift pressures

All parameters are defined in Figure 7-5. The uplift pressure across the base of the structural wedge usually varies from the undrained pressure head at the heel to the assumed reduced pressure head at the line of drains to the undrained pressure head at the toe, as shown in Figure 7-5. Uplift forces used for the sliding analyses should be selected in consideration of conditions which are presented in the applicable design memoranda. For a more detailed discussion of uplift under gravity dams, refer to EM 1110-2-2200, Gravity Dams.

g. Overturning. As stated previously, requirements for rotational equilibrium are not directly included in the general sliding stability equations. For some load cases, the vertical component of the resultant load will lie outside the kern of the base area, and a portion of the structural wedge will not be in contact with the foundation material. The sliding analysis should be modified for these load cases to reflect the following secondary effects due to coupling of sliding and overturning behavior.

(1) The uplift pressure on the portion of the base which is not in contact with the foundation material should be a uniform value which is equal to the maximum value of the hydraulic pressure across the base (except for instantaneous loads such as those due to seismic forces).

(2) The cohesive component of the sliding resistance should only include the portion of the base area which is in contact with the foundation material.

(3) The resultant of the lateral earth (soil) pressure is assumed to act at 0.38 of the wall height for horizontal or downward sloping backfills and at 0.45 of the wall height for upward sloping backfills.

(4) Cantilever or gravity walls on rock should be designed for at-rest earth pressures unless the foundation rock has an unusually low modulus.

7-11. Seismic Sliding Stability

The sliding stability of a structure for an earthquake-induced base motion should be checked by assuming the specified horizontal earthquake acceleration coefficient and the vertical earthquake acceleration coefficient, if included in the analysis, to act in the most unfavorable direction. The earthquake-induced forces on the structure and foundation wedges may then be determined by a quasi-static rigid body analysis. For the quasi-static rigid body analysis, the horizontal and vertical forces on the structure and foundation wedges may be determined by using the following equations:

$$H_{di} = M_i\ddot{X} + m_i\ddot{X} + H_i \tag{7-8}$$

$$V_{di} = Mg - m_i\ddot{y} \tag{7-9}$$

where

H_d = horizontal forces acting on the structure and/or wedge

V_d = vertical forces acting on the structure and or wedge

M = mass of the structure and/or wedge (weight/g)

m = added mass of reservoir and/or adjacent soil/rock

g = acceleration of gravity

$\ddot{X}$ = horizontal earthquake acceleration coefficient

$\ddot{y}$ = vertical earthquake acceleration coefficient

The subscript i, H, and V terms are as defined previously.

a. Earthquake acceleration. The horizontal earthquake acceleration coefficient can be obtained from seismic zone maps (ER 1110-2-1806) or, in the case where a design earthquake has been specified for the structure, an acceleration developed from analysis of the design earthquake. Guidance is being prepared for the latter type of analysis and will be issued in the near future; until then, the seismic coefficient method is the most expedient method to use. The vertical earthquake acceleration is normally neglected but can be taken as two-thirds of the horizontal acceleration if included in the analysis.

b. Added mass. The added mass of the reservoir and soil can be approximated by Westergaard's parabola (EM 1110-2-2200) and the Mononobe-Okabe method (EM 1110-2-2502), respectively. The structure should be designed for a simultaneous increase in force on one side and decrease on the opposite side of the structure when such can occur.

c. Analytical procedures. The analytical procedures for the seismic quasi-static analyses follows the procedures outlined in paragraphs 7-9*a* and 7-9*b* for the general wedge and alternate methods, respectively. However, the H_d and V_d terms are substituted for the H and W terms, respectively, in Equations 7-3 and 7-5a.

7-12. Factor of Safety

For major concrete structures (dams, lockwalls, basin walls which retain a dam embankment, etc.) the minimum required factor of safety for normal static loading conditions is 2.0. The minimum required factor of safety for seismic loading conditions is 1.3. Retaining walls on rock require a safety factor of 1.5; refer to EM 1110-2-2502 for a discussion of safety factors for floodwalls. Any relaxation of these values will be allowed only with the approval of CECW-E and should be justified by comprehensive foundation studies of such nature as to reduce uncertainties to a minimum.

Section III
Treatment Methods

7-13. General

Frequently a sliding stability assessment of structures subjected to lateral loading results in an unacceptably low factor of safety. In such cases, a number of methods are available for increasing the resistance to sliding. An increase in sliding resistance may be achieved by one or a combination of three mechanistic provisions. The three provisions include: increasing the resisting shear strength by increasing the stress acting normal to the potential failure surface; increasing the passive wedge resistance; and providing lateral restraining forces.

7-14. Increase in Shear Strength

The shear strength available to resist sliding is proportional to the magnitude of the applied stress acting normal to the potential slip surface. An increase in the normal stress may be achieved by either increasing the vertical load applied to the structural wedge and/or passive wedge(s) or by a reduction in uplift forces. The applied vertical load can be conveniently increased by increasing the mass of the structure or placing a berm on the downstream passive wedge(s). Installation of foundation drains and/or relief wells to relieve uplift forces is one of the most effective methods by which the stability of a gravity hydraulic structure can be increased.

7-15. Increase in Passive Wedge Resistance

Resistance to sliding is directly influenced by the size of the passive wedge acting at the toe of the structure. The passive wedge may be increased by increasing the depth the structure is embedded in the foundation rock or by construction of a key. Embedment and keys are also effective in transferring the shear stress to deeper and frequently more competent rock.

7-16. Lateral Restraint

Rock anchors inclined in the direction of the applied shear load provide a force component which acts against the applied shear load. Guidance for the design of anchor systems is discussed in Chapter 9 of this manual.

Chapter 8
Cut Slope Stability

8-1. Scope

This chapter provides guidance for assessing the sliding stability of slopes formed by excavations in rock or of natural rock slopes altered by excavation activities. Typical examples of slopes cut in rock include: foundation excavations; construction of project access roads; and development of dam abutments, spillways, and tunnel portals. This chapter is divided into three sections according to the general topic areas of modes of failure, methods of assessing stability, and treatment methods and planning considerations.

Section I
Modes of Failure

8-2. General

The primary objectives of any rock excavation is to minimize the volume of rock excavated while providing an economical and safe excavation suitable for its intended function. The objectives of economy and safety, as a rule, involve the maximization of the angle of inclination of the slope while assuring stability. Stability assurance requires an appreciation for the potential modes of failure.

8-3. Types of Failure Modes

Because of its geometry, rock slopes expose two or more free surfaces. Thus, as a rule, constituent rock blocks contained within the rock mass have a relative high kinematic potential for instability. In this respect, the type of failure is primarily controlled by the orientation and spacing of discontinuities within the rock mass as well as the orientation of the excavation and the angle of inclination of the slope. The modes of failure which are controlled by the above factors can be divided into three general types: sliding, toppling, and localized sloughing. Each type of failure may be characterized by one or more failure mechanisms.

8-4. Sliding Failure Modes

Figure 8-1 illustrates seven failure mechanisms that may be associated with the sliding failure mode. While other failure mechanisms are conceptually possible, the seven mechanisms illustrated are representative of those mechanisms most likely to occur. The following discussions provide a brief description of the conditions necessary to initiate each of the sliding mechanisms.

a. Single block/single sliding plane. A single block with potential for sliding along a single plane (Figure 8-1a) represents the simplest sliding mechanism. The mechanism is kinematically possible in cases where at least one joint set strikes approximately parallel to the slope strike and dips toward the excavation slope. Failure is impending if the joint plane intersects the slope plane and the joint dips at an angle greater than the angle of internal friction (ϕ) of the joint surface.

b. Single block/stepped sliding planes. Single block sliding along stepped planes (Figure 8-1b) is possible in cases where a series of closely spaced parallel joints strike approximately parallel to the excavation slope strike and dip toward the excavation slope. The parallel joints may or may not be continuous. However, at least one joint plane must intersect the slope plane. In the case of continuous parallel joints, a second set of joints is necessary. This second joint set must also strike more or less parallel to the slope and the magnitude and direction of the joint dip angle must be such that the joint plane does not intersect the slope plane.

c. Multiple blocks/multiple sliding planes. Multiple blocks, sliding along multiple planes (Figure 8-1c) is the most complicated planar type of sliding. The mechanism is associated with two or more joint sets that strike approximately parallel to the slope strike and dip in the direction of the excavation slope. At least one of the joint planes must intersect the excavated slope plane. For a failure to occur, the dip angle of the joint defining the base of the upper most block must be greater than the friction angle of the the joint surface. Furthermore, additional joints must be present which also strike approximately parallel to the strike of the excavated slope. These additional joints must either dip in a near vertical direction or dip steeply away from the slope plane.

d. Single wedge/two intersecting planes. Single wedge sliding (Figure 8-1d) can occur in rock masses with two or more sets of discontinuities whose lines of intersection are approximately perpendicular to the strike of the slope and dip toward the plane of the slope. In addition, this mode of failure requires that the dip angle of at least one joint-intersect is greater than the friction angle of the joint surfaces and that the line of joint intersection intersects the plane of the slope.

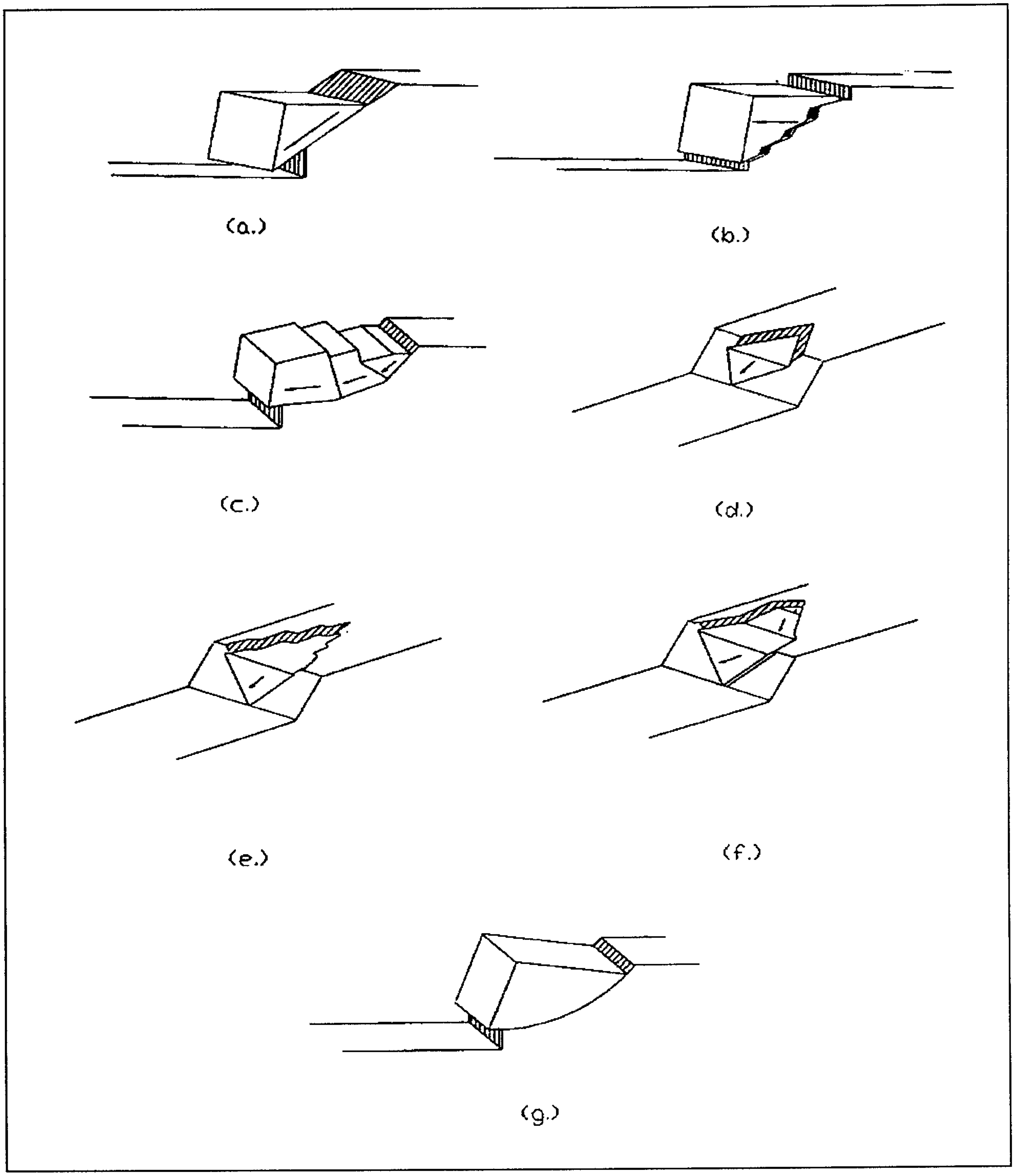


Figure 8-1. Failure mechanisms for the sliding failure mode: a) single block with single plane; b) single block with stepped planes; c) multiple blocks with multiple planes; d) single wedge with two intersecting planes; e) single wedge with multiple intersecting planes; f) multiple wedges with multiple intersecting planes; and g) single block with circular slip path

e. Single wedge/multiple intersecting planes. The conditions for sliding of a single wedge formed by the intersections of at least two discontinuity sets with closely spaced joints (Figure 8-1e) are essentially the same as discussed in paragraph 8-4*d.* above.

f. Multiple wedges/multiple intersecting planes. Multiple wedges can be formed by the intersection of four or more sets of discontinuities (Figure 8-1f). Although conceptually possible, the sliding failure of a multiple wedge system rarely occurs because of the potential for kinematic constraint.

g. Single block/circular slip path. Single block sliding failures along circular slip paths are commonly associated with soil slopes. However, circular slip failures may occur in highly weathered and decomposed rock masses, highly fractured rock masses, or in weak rock such as clay shales and poorly cemented sandstones.

8-5. Toppling Failure Mode

Toppling failure involves overturning or rotation of rock layers. Closely spaced, steeply dipping discontinuity sets that dip away from the slope surface are necessary prerequisites for toppling. In the absence of cross jointing, each layer tends to bend downslope under its own weight thus generating flexural cracks. If frequent cross joints are present, the layers can topple as rigid columns. In either case, toppling is usually initiated by layer separation with movement in the direction of the excavation. Layer separation may be rapid or gradual. Rapid separation is associated with block weight and/or stress relief forces. Gradual separation is usually associated with environmental processes such as freeze/thaw cycles.

8-6. Sloughing Failure Mode

Sloughing failures are generally characterized by occasional rock falls or localized slumping of rocks degraded by weathering. Rock falls occur when rock blocks become loosened and isolated by weathering and erosion. Some rocks disintegrate into soil-like material when exposed to repeated wetting and drying cycles. This material can fail in a fashion similar to shallow slump type failures commonly associated with soil slopes. Both rock falls and localized slumping constitute more of a maintenance problem than a major slope instability threat. However, slopes in sedimentary rock that are interbedded with shale layers can experience major slope failures initiated by localized deterioration of the shale layers. Deterioration of the shale layers leads to the undermining and hence failure of the more competent overlying layers.

8-7. Additional Factors Influencing Slope Stability

The geometric boundaries imposed by the orientation, spacing and continuity of the joints, as well as the free surface boundaries imposed by the excavation, define the modes of potential failure. However, failure itself is frequently initiated by additional factors not related to geometry. These factors include erosion, ground water, temperature, in-situ stress, and earthquake-induced loading.

a. Erosion. Two aspects of erosion need to be considered. The first is large scale erosion, such as river erosion at the base of a cliff. The second is relatively localized erosion caused by groundwater or surface runoff. In the first type, erosion changes the geometry of the potentially unstable rock mass. The removal of material at the toe of a potential slide reduces the restraining force that may be stabilizing the slope. Localized erosion of joint filling material, or zones of weathered rock, can effectively decrease interlocking between adjacent rock blocks. The loss of interlocking can significantly reduce the rock mass shear strength. The resulting decrease in shear strength may allow a previously stable rock mass to move. In addition, localized erosion may also result in increased permeability and ground-water flow.

b. Ground water. Ground water occupying the fractures within a rock mass can significantly reduce the stability of a rock slope. Water pressure acting within a discontinuity reduces the effective normal stress acting on the plane, thus reducing the shear strength along that plane. Water pressure within discontinuities that run roughly parallel to a slope face also increase the driving forces acting on the rock mass.

c. Temperature. Occasionally, the effects of temperature influence the performance of a rock slope. Large temperature changes can cause rock to spall due to the accompanying contraction and expansion. Water freezing in discontinuities causes more significant damage by loosening the rock mass. Repeated freeze/thaw cycles may result in gradual loss of strength. Except for periodic maintenance requirements, temperature effects are a surface phenomenon and are most likely of little concern for permanent slopes. However, in a few cases, surface deterioration could trigger slope instability on a larger scale.

d. State of stress. In some locations, high in-situ stresses may be present within the rock mass. High horizontal stresses acting roughly perpendicular to a cut slope

may cause blocks to move outward due to the stress relief provided by the cut. High horizontal stresses may also cause spalling of the surface of a cut slope. Stored stresses will most likely be relieved to some degree near the ground surface or perpendicular to nearby valley walls. For some deep cuts, it may be necessary to determine the state of stress within the rock mass and what effects these stresses may have on the cut slope.

Section II
Methods for Assessing Stability

8-8. General

This section presents a brief review of some of the more commonly used methods for assessing the stability of slopes cut in rock masses. The method selected for analyses depends upon the potential failure mode and, to some extent, the preference of the District Office responsible for the analyses. In this respect, the discussions will be divided according to potential failure modes. The potential failure modes include sliding, toppling, and localized sloughing. A detailed discussion of each of the various methods is beyond the scope of this manual. Hoek and Bray (1974), Canada Centre for Mineral and Energy Technology (1977a), Kovari and Fritz (1989) and Hendron, Cording and Aiyers (1980) provide general discussions on analytical methods for accessing the stability of rock slopes. Specific references are given which provide in depth details for each of the methods as they are discussed.

8-9. Sliding Stability Analyses

The majorities of the methods used in analyzing the sliding stability of slopes cut into rock masses are based on the principles of limit equilibrium. The mathematical formulation of the various methods depends upon the three general modes of sliding failure illustrated in Figure 8-1. These three general modes include planar slip surfaces, three-dimensional wedge shaped slip surfaces, and circular slip surfaces. Since the majority of sliding stability problems are indeterminate, a number of assumptions must be made about the location, orientation, and possible magnitude of the forces involved in the analysis. Different methods are presented below along with a short description of the assumptions that are made as well as the general procedure used for the analyses.

a. Planar slip surfaces. The analyses of planar slip surfaces assume that stability can be adequately evaluated from two-dimensional considerations. The following discussions summarize a number of different methods for analyzing the stability of planar slip surfaces. The methods are not all inclusive but rather are representative of commonly used methods that are currently available.

(1) Simple plane method. The simple plane method is applicable to slopes in which the potential slip surface is defined by a single plane, as illustrated in Figure 8-1a. The method is based on equilibrium between driving and resisting forces acting parallel and perpendicular to the potential slip surface. Mathematical expressions of the simple plane method can be found in most elementary physics text books. Convenient expressions are provided by Kovari and Fritz (1989).

(2) Two-dimensional wedge method. The two-dimensional wedge is suited for cases in which the potential failure surface of a rigid rock mass can be closely approximated by two or three planes. Hence, the method assumes that the potential failure mass can be divided into two or three two-dimensional wedges. A simplified approach assumes that forces between the wedges are horizontal. The horizontal force assumption generally results in a factor of safety that is within 15 percent (generally on the conservative side) of more accurate techniques which satisfy all conditions of equilibrium. Lambe and Whitman (1969) provide a detailed discussion and an example of the method.

(3) Generalized slip-surface methods for a rigid body. Generalized slip-surface methods refer to those methods which are used to solve two-dimensional rigid body stability problems using potential slip surfaces of any arbitrary shape. In this respect, the slip surfaces may be curvilinear in shape or defined by an assemblage of any number of linear segments as illustrated in Figure 8-1b. Of the available generalized slip-surface methods the two best known methods were proposed by Janbu (1954) and Morgenstern and Price (1965).

(a) Janbu's generalized slip-surface method is an iterative procedure using vertical slices and any shape slip-surface. The procedure, in its rigorous form, satisfies all conditions of equilibrium to include vertical and horizontal force equilibrium, moment equilibrium of the slices, and moment equilibrium of the entire slide mass. Complete equilibrium requires the solution of both shear and normal forces acting between slices. In the solution for the side forces Janbu's method assumes the point of side force application as well as the line of action of all the side forces. Janbu (1973) provides a detailed discussion of theory and application.

(b) Morgenstern and Price's generalized slip-surface method is similar to Janbu's method in that the procedure incorporates the interaction between a number of vertical slices. Complete equilibrium is achieved by assuming the values of variable ratios between the shear and normal forces acting on the sides of each slice. Morgenstern and Price (1965) provide a detailed discussion of the method.

(4) Generalized slip-surface methods for two or more rigid bodies. Generalized slip-surface methods for two or more rigid bodies refer to those analytical methods used to solve two-dimensional stability problems. In this special case, sliding can occur along the base of each body as well as between each body as illustrated in Figure 8-1c. At least three methods are available for analyzing this special case. These three methods include methods proposed by Kovari and Fritz (1989) and Sarma (1979) as well as the distinct element numerical model method (e.g. Cundall 1980).

(a) Kovari and Fritz's (1989) method provides a relatively simple solution for the factor of safety of two or more adjacent blocks subject to sliding. The potential slide surface along the base of each block is represented by a single plane. Blocks are separated by planes of discontinuity which may be inclined at arbitrary angles with respect to the base of the potential slide plane. The method satisfies force equilibrium. Moment equilibrium is not considered. In this respect, solutions for the factor of safety tend to be conservative.

(b) Sarma (1979) proposed a comprehensive solution to the two-dimensional, multiple block sliding problem which satisfies both moment and force equilibrium. The method utilizes slices that can be nonvertical with nonparallel sides. Solution for the factor of safety requires an iterative process. As such, from a practical point it is usually more convenient to program the method for use on programmable calculators or personal computers.

(c) The distinct element (e.g. Cundall 1980) method is based on equations of motion for particles or blocks. The method offers a useful tool for examining the phenomenology and kinematics of potentially unstable slopes.

b. Three-dimensional wedge shaped slip surfaces. The majority of potentially unstable rock slopes can be characterized as three-dimensional wedge problems as illustrated in Figure 8-1d, 8-1e, and 8-1f. The analytical analysis of three-dimensional problems is substantially simplified if the geotechnical professional responsible for the stability analysis is conversant with the use of stereographic projection. Stereographic projection allows convenient visualization of the problem being analyzed as well as the definition of geometric parameters necessary for analysis. Goodman (1976), Hoek and Bray (1974), and Priest (1985) provide detailed discussions of theory and application of stereographic projection techniques. Once the problem geometry has been defined, an analytical method can be selected for assessing the sliding stability of the slope. For convenience of discussion, methods for assessing sliding stability will be divided into two categories: methods for single three-dimensional wedges and methods for multiple three-dimensional wedges.

(1) Three-dimensional single wedge methods. Three-dimensional single wedge methods are applicable to slopes in which the potential instability is defined by a single rigid wedge as illustrated in Figures 8-1d and 8-1e. Sliding may occur along one or more planar surfaces. As a rule, analytical solutions for the factor of safety are based on the principles of limit equilibrium in which force equilibrium is satisfied. A large number of expressions for the solution of factors of safety are reported in the literature. Hendron, Cording, and Aiyer (1980), Hoek and Bray (1974), Kovari and Fritz (1989) provide expressions and detailed discussions of the method. Hendron, Cording, and Aiyer (1980) and Chan and Einstein (1981) also provide methods for addressing potential block rotation as well as transverse sliding.

(2) Three-dimensional, multiple wedge, methods. Although conceptually possible, multiple three-dimensional wedge systems seldom fail in sliding because of the potential for kinematic constraint. Generalized analytical solutions for the factor of safety in such cases are not readily available. In this respect, three-dimensional distinct element methods (Cundall 1980) offer a means of evaluating the kinematics of potentially unstable slopes.

c. Circular slip surfaces. As in planar slip surfaces, the analyses of circular slip surfaces assume that stability can be adequately evaluated from two-dimensional considerations as illustrated in Figure 8-1g. The methods are generally applicable to rock slopes excavated in weak intact rock or in highly fractured rock masses. Of the various circular slip surface methods available, two of the more commonly used include the ordinary method of slices and the simplified Bishop method.

(1) Ordinary method of slices. The ordinary method of slices (EM 1110-2-1902) is also known as the Swedish Circle Method or the Fellenious Method. In this method the potential sliding mass is divided into a number of vertical slices. The resultant of the forces acting on the

sides of the slices act parallel to the base of that particular slice. Only moment equilibrium is satisfied. In this respect, factors of safety calculated by this method are typically conservative. Factors of safety calculated for flat slopes and/or slopes with high pore pressures can be on the conservative side by as much as 60 percent, at least when compared with values from more exact solutions.

(2) Simplified Bishop method. The Simplified Bishop Method (Janbu et al. 1956) is a modification of a method originally proposed by Bishop (1955). In the simplified method, forces acting on the sides of any vertical slice is assumed to have a zero resultant in the vertical direction. Moment equilibrium about the center of the slip surface circle as well as force equilibrium are satisfied. There is no requirement for moment equilibrium of individual slices. However, factors of safety calculated with this method compare favorably with values obtained from more exact solution methods.

8-10. Toppling Stability Analyses

Two-dimensional considerations indicate that toppling can occur if two conditions are present. In this respect, toppling can occur only if the projected resultant force (body weight plus any additional applied forces) acting on any block of rock in question falls outside the base of the block and the inclination of the surface on which the block rests is less than the friction angle between the block and surface. However, in actual three-dimensions, rock slopes consist of a number of interacting blocks which restrict individual block movement. As a result the mechanism is likely to be a complex combination of sliding and toppling. Due to the complexities of failure, generalized analytical methods which attempt to solve for the factor of safety have not been developed. Three-dimensional numerical methods such as the distinct element method can, however, offer insight as to the kinematics of failure.

8-11. Localized Sloughing Analyses

Localized sloughing failures refer to a variety of potential failure modes. These modes can range from rotational failure of individual blocks to minor sliding failures of individual small blocks or mass of rock. These types of potential instability are frequently treated as routine maintenance problems and, as such, are seldom analyzed for stability.

8-12. Physical Modeling Techniques

In addition to the analytical methods, there exist a number of physical modeling techniques used for problems where analytical techniques may not be valid or may be too complex. Available methods include the Base Friction Model, Centrifuge Model, and small-scale models. All of these techniques have shortcomings in that basic parameters to include length, mass, and strength must be scaled. The difficulty arises in that all three parameters must be scaled in the same proportions. Simultaneous scaling requirements are difficult to achieve in practice. Therefore, it is common to scale the most important parameter(s) accurately and then attempt to relate the influence of the lesser important parameters to the test results. Physical modeling techniques are discussed by Hoek and Bray (1974) and Goodman (1976).

a. Base friction modeling. This modeling technique uses a frictional rolling base in the form of a long sheet or a conveyor-like belt that simulates gravity. The model material is typically a sand-flour-vegetable oil material that closely models friction angles of discontinuous rock. A two-dimensional model of the slope or excavation is formed on the table. As the belt moves, the model slowly deforms. The technique cannot be used to model dynamic loadings. It is an excellent method to investigate the kinematics of jointed two-dimensional systems.

b. Centrifuge modeling. Centrifuge modeling attempts to realistically scale body forces (i.e., gravitational forces). In this respect, centrifuge modeling may be a possible solution in cases where gravity plays an important role. Centrifuge methods are presently expensive and the available centrifuges typically have long waiting lists. Generally, these machines only allow rather small models to be evaluated. Also, instrumentation of these models is required as one cannot scrutinize the model during testing, except perhaps with the help of a visual aid.

c. Scaled models. These models are straightforward, however, they require model materials to build the scale model. The model material development is difficult due to the previously mentioned scaling problems. Use of heavy materials such as barite might be of some use in scaling gravitational effects. In addition scaling associated with modeling requirements, the scale effects associated with shear strength selection must be also be considered as discussed in Chapter 4 of this manual.

8-13. Design Considerations

A rock slope is accessed to be stable or potentially unstable depending upon the value of the calculated factor of safety. The calculated factor of safety is primarily dependent upon the geometry of the potential failure path selected for analyses and the shear strength representative of the potential failure surface. In addition, other factors, such as ground water conditions, potential for erosion, seismic loading, and possible blast-induced loosening of the rock mass must also be considered.

a. Factor of safety. For major rock slopes where the consequence of failure is severe, the minimum required calculated factor of safety is 2.0. For minor slopes, or temporary construction slopes where failure, should it occur, would not result in bodily harm or a major loss of property, the minimum required factor of safety is 1.3. The minimum required factor of safety for rock slopes subject to and assessed for seismic loading is 1.1. Any relaxation of these values will be allowed only with the approval of CECW-EG and should be justified by comprehensive studies of such a nature as to reduce uncertainties to a minimum.

b. Critical potential failure paths. For a given rock slope, a number of potential failure paths are kinematically possible. Each kinematically possible failure path must be analyzed. The critical potential failure path is that potential slip surface which results in the lowest value for the factor of safety. For a rock slope to be judged safe with respect to failure the factor of safety calculated for the critical potential failure path must be equal to or greater than the appropriate minimum required factor of safety.

c. Representative shear strength. Procedures for selecting appropriate shear strengths representative of potential failure paths are discussed in Chapter 4 of this manual.

d. Ground water conditions. Unlike natural rock slopes, cut slopes must be analyzed prior to excavation. Hence, while fluctuations in ground water levels may be known prior to design, the influence on these fluctuations due to excavation of a slope is difficult to predict. In this respect, assumptions pertaining to the phreatic surface and potential seepage pressures should be made on the conservative side.

e. Effects of erosion. Certain argillaceous rock types (e.g. some shales) are susceptible to erosion caused by slaking upon repeated wetting and drying cycles. Soft sedimentary rocks, in general, are also susceptible to erosion processes due to normal weathering, stream flow, or wave action. In this respect, stability analyses must either account for the effects of potential erosion (i.e. loss of slope toe support and/or undermining of more competent upper layers) or the overall design must provide provision to control the effects of erosion.

f. Seismic loading. Where applicable, the stability of rock slopes for earthquake induced base motion should be checked by assuming that the specified horizontal and vertical earthquake accelerations act in the most unfavorable direction. In this respect, earthquake-induced forces acting on a potentially unstable rock mass may be determined by a quasi-static rigid body approach in which the forces are estimated by Equations 7-8 and 7-9, as given in Chapter 7 of this manual.

g. Potential blast effects. Shear strengths selected for design analyses are generally based on preconstruction rock mass conditions. Rock slopes are commonly excavated by drill and blast techniques. If improperly used, these excavation techniques can significantly alter the material properties of the rock mass comprising the slope. These alterations are more commonly evident as loosened rock which results in a reduction of strength. Design analyses must either account for potential blast-induced loosening with subsequent loss of strength, or ensure that proper drill and blast procedures are used in the excavation process. Proper drill and blast procedures are given in EM 1110-2-3800.

Section III
Treatment Methods and Planning Considerations

8-14. General

The stability assessment of rock slopes frequently indicates an impending failure is possible. In such cases, a number of methods are available for improving the overall stability. An appreciation of the mechanics associated with rock slope stability together with an understanding of treatment methods for improving the stability of potentially unstable slopes permit the detailed planning and implementation of a slope stability program.

8-15. Treatment Methods

The available treatment methods include alteration of slope geometry, dewatering to increase resisting shear strength, rock anchors, and toe berms protection to prevent slaking and erosion effects.

a. Slope geometry. In the absence of an imposed load, the forces which tend to cause the instability of a slope are a direct function of both slope height and angle of inclination. A reduction of slope height and/or angle of inclination reduce the driving forces and, as a result, increase stability. In addition, since the majority of rock slope stability problems are three-dimensional in nature, a few degrees of rotation in the strike of the slope can, in some cases, cause a potentially unstable slope to become kinematically stable.

b. Dewatering. The presence of ground water within a rock slope can effectively reduce the normal stress acting on the potential failure plane. A reduction in normal stress causes a reduction in the normal stress dependent friction component of shear strength. Ground water induced uplift can be controlled by two methods, internal drains and external drains. In this respect, drainage is often the most economical and beneficial treatment method.

(1) Internal drains. Properly designed and installed internal drains can effectively reduce ground water levels within slopes thereby increasing stability. The specific design of an effective drain system depends upon the geohydraulic characteristics of the rock mass (i.e. joint spacing, condition and orientation, as well as source of ground water). As a minimum, a effective drain system must be capable of draining the most critical potential failure surface. In climates where the ground surface temperature remains below freezing for extended periods of time, the drain outlet must be protected from becoming plugged with ice. Hoek and Bray (1974) describe various types of internal drains.

(2) External drains. External or surface drains are designed to collect surface runoff water and divert it away from the slope before it can seep into the rock mass. Surface drains usually consist of drainage ditches or surface berms. Unlined ditches should be steeply graded and well maintained.

c. Rock anchors. Rock bolts, as well as, grouted in place reinforcement steel and cables are commonly used to apply restraining forces to potentially unstable rock slopes. Rock anchors may be tensioned or untensioned depending, primarily, upon the experience and preference of the District office in charge of design. It must be realized, however, that untensioned anchors rely on differential movement of the rock mass to supply the necessary resisting force and that very little cost is involved in tensioning. Where deformations must be minimized or where initial resisting forces must be assured, the tensioning of rock anchors upon installation may be required.

d. Erosion protection. Shotcrete, frequently with the addition of wire mesh and/or fibers, is an effective surface treatment used to control slaking and raveling of certain argillaceous rock types that can lead to erosion problems. The treatment also prevents loosening of the rock mass due to weathering processes and provides surface restraint between rock bolts.

e. Toe berms. Toe berms provide passive resistance that can be effective in improving the stability of slopes which the critical potential failure plane passes within close proximity to the toe of the slope.

8-16. Planning Considerations

With the design of numerous slopes or extremely long slopes, it is economically imperative that a system be followed which will eliminate naturally stable or noncritical slopes from study at a very early stage of investigation and allow concentration of effort and resources on those slopes which are critical. In this respect, a rock slope design flow chart which shows the steps required for design of rock slopes has been proposed by Hoek and Bray (1981) and is presented in Figure 8-2 with some modifications. The approach to the design of a slope is proposed in two phases.

a. Phase one. The first phase involves preliminary evaluations of available geologic data which may include air photo interpretations, surface mapping, and gathering of data from rock cores from boreholes. Preliminary stability studies are then conducted using estimates of shear strengths of the discontinuities from index tests, experience, and from back analyses of existing slope failures in the area. These preliminary studies should identify those slopes which are obviously stable and those in which there are some risks of failure. Slopes which are proven to be stable from the preliminary analysis can be designed on the basis of operational considerations.

b. Phase two. Those slopes proved to have a risk of failure require further analyses based upon more detailed information of geology, ground water, and mechanical properties of the rock mass. These analyses should consider the widest possible range of conditions which affect the stability of the slope. Slopes which are shown by detailed analyses to have an unacceptably high risk of failure must be redesigned to include stabilization measures. The operational and cost benefits of the

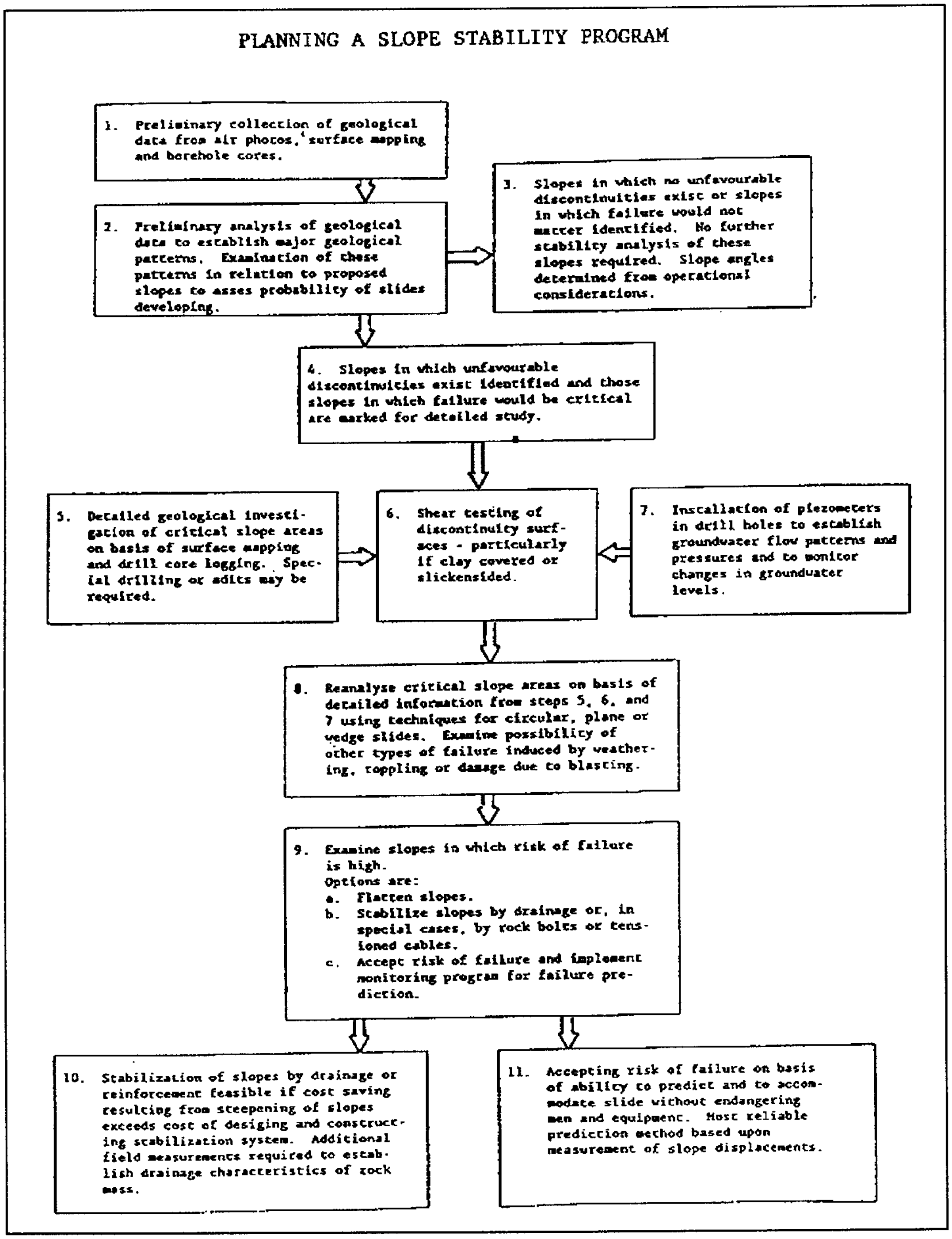


Figure 8-2. Analysis of the stability of slopes (modified from Hoek and Bray 1981)

stabilization measures should be compared with their implementation cost to determine the optimum methods of stabilization. The risk of failure for some slopes may be considered acceptable if slope monitoring would allow failures to be predicted in advance and if the consequences of a failure can be made acceptable.

Chapter 9
Anchorage Systems

9-1. Scope

This chapter provides guidance for the design and evaluation of anchor systems used to prevent the sliding and/or overturning of laterally loaded structures founded on rock masses. This chapter supplements guidance provided in EM 1110-1-2907. The chapter is divided into two sections: Modes of Anchor-Rock Interaction and Methods of Analyses.

Section I
Modes of Anchor-Rock Interaction

9-2. General

Anchor systems may be divided into two general categories--tensioned and untensioned. The primary emphasis in the design, or selection of an anchorage system, should be placed on limiting probable modes of deformation that may lead to failure or unsatisfactory performance. The underlying premise of anchorage is that rock masses are generally quite strong if progressive failure along planes of low strength can be prevented. Both tensioned and untensioned anchors are suitable for the reduction of sliding failures in, or on, rock foundations. Tensioned anchor systems provide a means for prestressing all, or a portion, of a foundation, thus, minimizing undesirable deformations or differential settlements. Preconsolidation of rock foundations results in joint closure and what appears as strain hardening in some foundations.

9-3. Tensioned Anchor Systems

A typical prestressed anchorage system is shown in Figure 9-1. The use of grouted anchorages is practically universal, particularly with high capacity tendon systems. Upon tensioning, load is transferred from the tensioning element, through the grout, to the surrounding rock mass. A zone of compression is established (typically assumed as a cone) within the zone of influence. Tensioned anchor systems include rock bolts and rock anchors, or tendons. The following definitions are as given in EM 1110-1-2907.

a. Rock bolt. A tensioned reinforcement element consisting of a rod, a mechanical or grouted anchorage, and a plate and nut for tensioning or for retaining tension applied by direct pull or by torquing.

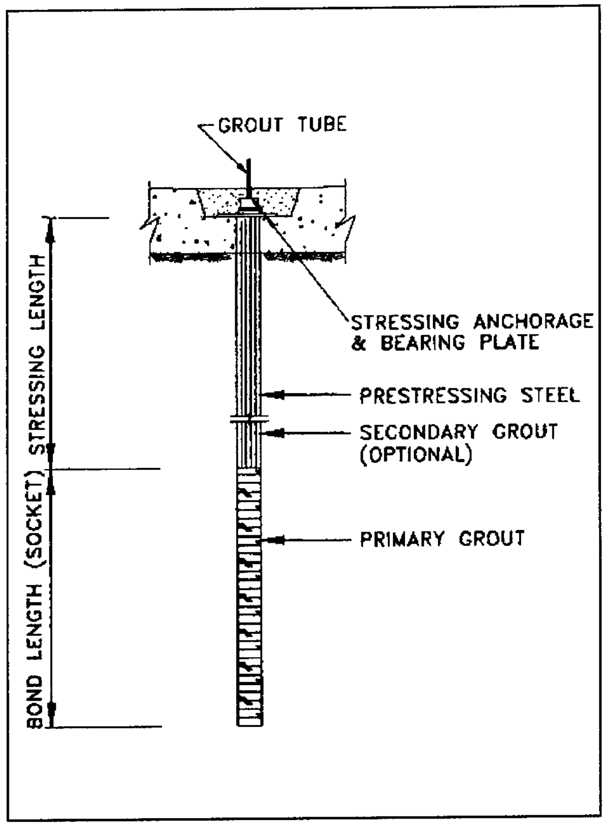


Figure 9-1. Typical components of a tensioned rock anchor (from EM 1110-1-2907)

b. Prestressed rock anchor or tendon. A tensioned reinforcing element, generally of higher capacity than a rock bolt, consisting of a high strength steel tendon (made up of one or more wires, strands, or bars) fitted with a stressing anchorage at one end and a means permitting force transfer to the grout and rock at the other end.

9-4. Untensioned Anchor Systems

Untensioned rock anchors are generally referred to as rock dowels and are defined in EM 1110-1-2907 as an untensioned reinforcement element consisting of a rod embedded in a mortar or grout filled hole. Dowels provide positive resistance to dilation within a rock mass and along potentially unstable contact surfaces. In addition to the development of tensile forces resisting dilation, passive resistance against sliding is developed within a rock mass when lateral strains occur. The interaction between the dowel and the rock mass is provided through the cohesion and friction developed along the grout column which bonds the rod and the rock. Untensioned anchor systems should not be used to stabilize gravity structures.

Section II
Methods of Analysis

9-5. General

Typically, analyses of systems used to anchor mass concrete structures consist of one of two methods: procedures based upon classical theory of elasticity or procedures based upon empirical rules or trial and error methods. The gap between the methods has been narrowed by research in recent years but has not significantly closed to allow purely theoretical analysis of anchor systems. The following discussions on methods of analyses are divided into tensioned and untensioned anchor systems.

9-6. Analyses for Tension Anchor Systems

The design and analysis of anchor systems include determination of anchor loads, spacing, depth, and bonding of the anchor. Safety factors are determined by consideration of the following failures; within the rock mass, between the rock and grout/anchor, between the grout and the tendon or rod, and yield of the tendon or top anchorage.

a. Anchor loads. Anchor loads for prestressed tensioned anchors are determined from evaluation of safety factor requirements of structures. Anchors may be designed for stability considerations other than sliding to include overturning and uplift. Other factors must also be considered. However, anchor forces required for sliding stability assurance typically control design. Procedures for determining anchor forces necessary for stability of concrete gravity structures are covered in EM 1110-2-2200.

b. Anchor depths. Anchor depths depend upon the type of rock mass into which they are installed and the anchor pattern (i.e., single anchor, single row of anchors, or multiple rows of anchors). The anchor depth is taken as the anchor length necessary to develop the anchor force required for stability. The entire anchor depth lies below the critical potential failure surface.

(1) Single anchors in competent rock. The depth of anchorage required for a single anchor in competent rock mass containing few joints may be computed by considering the shear strength of the rock mobilized around the surface area of a right circular cone with an apex angle of 90 degrees (see Figure 9-2a). If it is assumed that the in-situ stresses as well as any stresses imposed on the foundation rock by the structure is zero, then the shear strength can be conservatively estimated as equal to the rock mass cohesion. In such cases the anchor depth can be estimated from Equation 9-1.

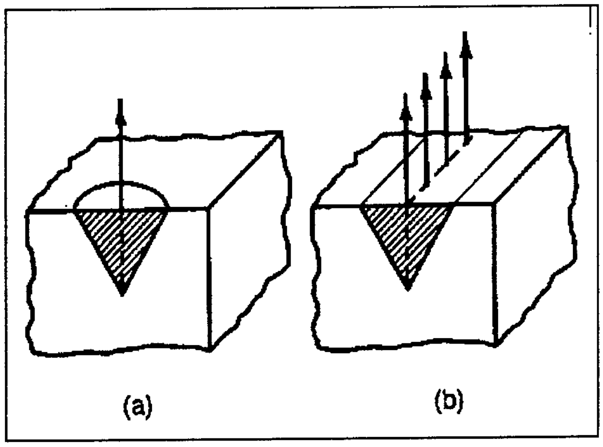


Figure 9-2. Geometry of rock mass assumed to be mobilized at failure (a) individual anchor in isotropic medium and (b) line of anchors in isotropic medium (after Littlejohn 1977)

$$D = [(FS)\ (F) / c\ \pi]^{1/2} \tag{9-1}$$

where

D = the required depth of anchorage

FS = the appropriate factor of safety

c = the rock mass cohesion intercept

F = the anchor force required for stability

(2) Single row of anchors in competent rock. The depth of anchorage for a single row of anchors (see Figure 9-2b) installed in competent rock and spaced a distance s apart may be computed as follows:

$$D = \frac{(FS)\ (F)}{cs} \tag{9-2}$$

where

F = the anchorage force on each anchor

All other parameters are as previously defined.

(3) Multiple rows of anchors in competent rock. For a multiple row of anchors with rows spaced a distance ℓ apart, typically, only the weight of the rock mass affected is used in calculations of resisting force. Under this assumption, the depth of anchorage required to resist a anchorage force F per anchor is computed as follows:

$$D = \frac{(FS)\ (F)}{\gamma\ \ell\ s} \qquad (9\text{-}3)$$

where γ = the unit weight of the rock. All other parameters are as previously defined.

(4) Single anchor in fractured rock. In fractured rock, the strength of the rock mass subjected to a tensile force (the anchor force) cannot typically be relied upon to provide the necessary resistance. For this reason, only the weight of the affected one is considered. Based upon this assumption, the depth of anchorage is completed as follows:

$$D = \left(\frac{3\,(FS)\,(F)}{\gamma\ \pi}\right)^{1/3} \qquad (9\text{-}4)$$

where γ = the unit weight of the rock. All other parameters are as previously defined.

(5) Single row of anchors in fractured rock. As in the case of a single anchor in fractured rock, typically only the weight of the affected wedge of rock is relied upon to provide the necessary resistance. Hence, for a single row of anchors in fractured rock spaced S distance apart, the anchorage depth is computed as follows:

$$D = \left(\frac{(FS)(F)}{\gamma\ S}\right)^{1/2} \qquad (9\text{-}5)$$

All other parameters are as previously defined.

(6) Multiple rows of anchors in fractured rock. For multiple rows of anchors with rows spaced ℓ distance apart, again only the weight of the affected rock mass resists the anchor force. In this respect Equation 9-3 is valid.

c. Anchor bonding. The above equations, presented for analysis of anchor system, assume sufficient bond of the anchor to the rock such that failures occur within the rock mass. The use of grouted anchorages has become practically universal with most rock reinforcement systems. The design of grouted anchorages must, therefore, insure against failure between the anchor and the grout, as well as, between the grout and the rock. Experience and numerous pull-out tests have shown that the bond developed between the anchor and the grout is typically twice that developed between the grout and the rock. Therefore, primary emphasis in design and analysis is placed upon the grout/rock interface. For straight shafted, grouted anchors, the anchor force which can be developed depends upon the bond stress, described as follows:

$$F = \pi d L \tau \qquad (9\text{-}6a)$$

$$\tau = 0.5\tau_{ult} \qquad (9\text{-}6b)$$

where

d = the effective diameter of the borehole

L = length of the grouted portion of the anchor bond length (normally not less than 10 ft)

τ = the working bond strength

τ = the ultimate bond strength at failure

Values of ultimate bond strength are normally determined from shear strength data, or field pull-out tests. In the absence of such tests, the ultimate bond stress is often taken as 1/10 of the uniaxial compressive strength of the rock or grout (whichever is less) (Littlejohn 1977) up to a maximum value of 4.2 MPa (i.e., 600 psi).

9-7. Dowels

Structures should in principle be anchored, when required, to rock foundations with tensioned or prestressed anchorage. Since a displacement or partial shear failure is required to activate any resisting anchorage force, analysis of the contribution of dowels to stability is at best difficult. Dilation imparts a tensile force to dowels when displacements occur over asperities but the phenomenon is rarely quantified for analytical purposes.

9-8. Design Considerations

a. Material properties. The majorities of material properties required for the design of anchor systems are also typically required for the investigation of other aspects of the foundation design. The selection of

appropriate material properties is discussed in Chapter 4 of this manual. Design anchor force derived from calculations not associated with sliding instability must consider the buoyant weight of rock where such rock is submerged below the surface water or ground water table. Tests not necessarily considered for typical foundation investigations but needed for anchor evaluations include rock anchor pull-out tests and chemical tests of the ground water. Rock anchor pull-out tests (Rock Testing Handbook, RTH 323) provide valuable data for determining anchorage depth and anchor bond strength. Hence, a prudent design dictates that pull-out tests be performed in the rock mass representative of the foundation conditions and anticipated anchor depths. Ground water chemical tests establish sulphate and chloride contents to be used as a guide in designing the anchor grout mix. In addition, the overall corrosion hazard for the anchor tendon steel should be established by chemical analysis. Such analyses are used to determine the amount and type of corrosion protection required for a particular foundation.

b. Factors of safety. The appropriate factor of safety to be used in the calculations of anchor force and anchorage depth must reflect the uncertainties and built-in conservatism associated with the calculation process. In this respect, anchor force calculations should be based on the factor of safety associated with sliding stability of gravity structures discussed in Chapter 7. Anchorage depth calculations based on the unit weight of the rock mass (Equations 9-3, 9-4, and 9-5) should use a minimum factor of safety of 1.5. All other anchorage depth calculations (i.e., Equations 9-1 and 9-2) should use a minimum factor of safety of 4.0 unless relaxed by CECW-EG for special circumstances.

c. Total anchor length. In addition to the anchor depth and anchor bonding considerations given by Equations 9-1 to 9-5 and Equation 9-6, respectively, the total anchor length (L_τ) is controlled by the location at which the rock mass is assumed to initiate failure should a general rock mass failure occur. Littlejohn and Bruce (1975) summarize the assumed location of failure initiation commonly used in practice. As indicated in Figure 9-3, three locations are commonly assumed: potential failure initiates at the base of the socket; potential failure initiates at the midpoint of the socket; or potential failure initiates at the top of the socket. The implication with respect to the total anchor length imposed by each failure location assumption is as shown in Figure 9-3. For the design of anchors in competent or fractured rock masses where the bond length is supported by pull-out tests, the potential for rock mass failure is assumed to initiate at the base of the anchor as shown in Figure 6-3a. For preliminary design where pull-out tests are not yet available or in highly fractured and very weak material, such as clay shale, the potential for failure is assumed to initiate at the midpoint of the socket as shown in Figure 6-3b. However, in the case of highly fractured and very weak material, pull-out tests must be performed to verify that the bond length is sufficient to develop the ultimate design load as specified in EM 1110-2-2000. Any relaxation in total anchor length requirements must be approved by CECW-EG.

d. Corrosion protection. The current industry standard for post-tensioned anchors in structures requires double corrosion protection for all permanent anchors.

e. Design process. The rock anchor design process is conveniently divided into two phases; the initial design phases and the final detailed phase. Additional details are provided in EM 1110-2-2200 and Post-Tensioning Institute (1986).

(1) Initial phase. The design process is initiated by an evaluation which finds that a given structure is potentially unstable without additional restraining forces. If the potential instability is due to potential for sliding, the magnitude of restraining forces is calculated according to procedures given in EM 1110-2-2200. Restraining forces necessary to control other modes of potential instability, such as overturning, uplift pressures, or excessive differential deformations are determined on a case-to-case basis. The magnitude of the required restraining force is evaluated with respect to the economics and practicality of using rock anchors to develop the necessary force.

(2) Final phase. The final detailed design phase is a trial and error process which balances economic and safety considerations with physical consideration of how to distribute the required restraining force to the structure and still be compatible with structure geometry and foundation conditions. While sequential design steps reflect the preference of the District Office, general design constraints usually dictate that the total restraining force be divided among a number of anchors. The number of anchors and hence the spacing between anchors and anchor rows, as well as the anchor orientation and installation details, are controlled by the geometry of the structure. Foundation conditions control the anchorage depth as well as the amount and type of corrosion protection. Anchor depths between adjacent anchors should be varied in order to minimize adverse stress concentrations.

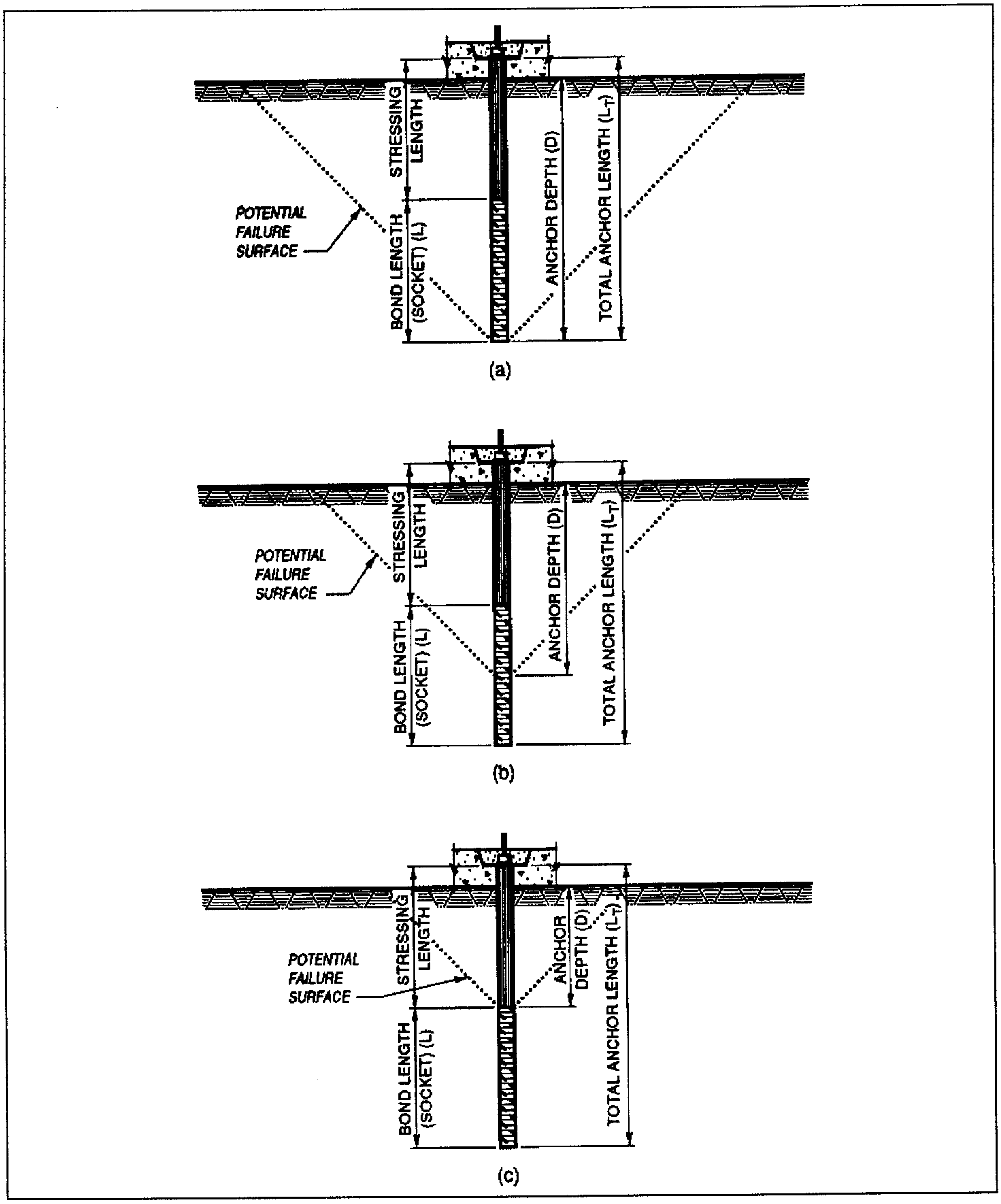


Figure 9-3. Potential failure surfaces commonly assumed for the design of anchor depths in rock masses: (a) potential failure initiates at the base of the socket; (b) potential failure initiates at the midpoint of the socket; (c) potential failure initiates at the top of the socket

Chapter 10
Instrumentation

10-1. Scope

This chapter provides general guidance for the selection and use of instrumentation to monitor cut slopes such as might be necessary for the construction of rock foundations and roads as well as structures founded on rock such as dams, lock walls, and retaining structures. Instrumentation for monitoring ground vibrations, water levels, and pore-water pressure measurements are discussed in more detail than other instrumentation because of their widespread use. The limitations as well as data interpretation and evaluation considerations are also discussed. Detailed descriptions and installation considerations, of the various types of instrumentation discussed herein, can be found in the referenced publications. The chapter is divided into four sections as follows: Planning Considerations; Typical Applications; Types of Instruments; and Data Interpretation and Evaluation.

Section I
Planning Considerations

10-2. General

Instrumentation is necessary on a project to assure that design criteria are being met, thereby assuring the safety of the structure, gain information valuable to future project design, monitor suspected problem areas to determine safety and remedial measures required, and monitor effectiveness of remedial measures.

10-3. Program Initiation

An instrumentation program should be planned during the design of a project. The specific areas and phases of the project from which data need to be gathered are determined using the rock mechanics analyses and models discussed in previous chapters. In order to obtain the most complete picture of how a rock mass is responding to the construction and operation of a project, instrumentation should be installed where possible before or during construction. Early installation rarely increases the cost of the instrumentation program, but does require more planning.

10-4. Cost Control

The instrumentation program should be well planned to assure that all necessary data will be collected and that excessive costs are not incurred. The main expenses of an instrumentation program include instrument purchase, installation, maintenance, data gathering, and data interpretation. Excessive costs in each of these areas are incurred if instrument types and placement are planned unwisely leading to more instrumentation than is necessary for the intended purpose or difficulty in interpreting data due to lack of information. The instrumentation program must be flexible enough to allow for changes necessary due to actual conditions encountered during construction.

10-5. Types and Number of Instruments

The parameters which are most often measured are deformation, load/stress, pore-water pressures and water levels, and ground vibrations. The types of instrumentation used to measure these parameters are listed in Table 10-1. The number of instruments and various types that will be required on a specific project are dependent on the purpose of the structure and the geologic conditions. The instrumentation program for every project should be designed specifically for that project and the expected conditions and should use the principles of rock mechanics. Rock instrumentation must reflect conditions over a large area of rock. Measurements made over small areas will yield data so influenced by small random features that it will be meaningless. Great care should be taken to assure that the particular instrumentation used will yield the type of information required at the necessary accuracy. An instrumentation program should be kept as simple as possible and still meet the objectives of the

Table 10-1
Types of Rock Foundation Instruments

Deformation	Load/Stress	Pore-Water Pressure	Ground Vibration
Surveying Inclinometers Extensometers Settlement Indicators Heave Points	Load Cells Piezometers Uplift Pressure Cells	Piezometers	Seismographs

program. A complicated instrument is generally harder to maintain and less reliable than a simple type. Simple, direct measurements are most easily and quickly interpreted.

Section II
Applications

10-6. General

This section describes some of the more common applications of rock mechanics instrumentation. The discussions are divided into two general topic areas related to project features addressed in this manual. These two topic areas include cut slope instrumentation and structure/foundation instrumentation.

10-7. Cut Slope Instrumentation

The number, types, and location of instruments used in cut slopes are highly dependent on the cut configuration, the geologic conditions that are involved, and the consequence should a failure occur. As a rule, however, instrumentation associated with cut slopes can be grouped into instruments used to make surface measurements and those used to make subsurface measurements.

a. Surface measurements. Surface measurement instruments are primarily used to measure surface deformations. Since surface instrumentation reveals little as to underlying mechanisms causing deformation, the instrumentation is used to detect new areas of distress or precursor monitoring of rock masses subject to impending failure. The degree of precision required by the intended purpose of instrumentation dictates the type of instrument used to measure deformation.

(1) Surveying. If the slope is stable, then periodic surveying of the floor and sidewalls using permanent monuments and targets may be the only instrumentation required. Precise, repetitive surveying of a network of such survey points is a relatively inexpensive method of detecting slope movement, both vertical and lateral. When a problem is detected, surveying can be used to define the area of movement. Evaluation of problem areas is required to determine if additional instrumentation is required. Depending on other factors, surveying may be continued, perhaps with increasing frequency, until remedial measures appear to be inevitable. In other cases, the failure of the slope may be more acceptable than the cost of the remedial measures and surveying would be continued until the slope failed, to insure the safety of personnel and equipment when failure occurs. Details of the instruments and surveying methods used may be found in TM 5-232, "Elements of Surveying" and TM 5-235, "Special Surveys."

(2) Surface deformation. In most cases, however, additional instrumentation will be required to provide the information which enables the investigator to find or to define the causes of the movement and to monitor the rate of movement. Tension cracks which appear at the crest of a slope or cut face may be monitored by surface type extensometers. This type of extensometer generally consists of anchor points installed on either side of the zone to be monitored. The zone may be one joint or crack or several such features. A tape or bar, usually composed of invar steel, is installed between the anchor points. A Newcastle extensometer may be installed on the tape to allow for very accurate readings which are necessary to measure the small initial indications of movement. For measuring larger movements, which would occur later and when continuous measurements are required, a bar and linear potentiometer can be installed between the stakes. See Chapter 8 of the Canada Centre for Mineral and Energy Technology (1977b) for details. If very large measurements are expected, a simple inexpensive system, which uses a calibrated tape to measure the change in distance between the two anchor points should be used. The tape can be removed after a reading is made. This instrument aids in the determination of the surface displacement of individual blocks and differential displacements within an unstable zone. Dunnicliff (1988) provides an excellent review of the various types of surface monitored extensometers.

b. Subsurface measurements. Subsurface instrumentation provides greater detail of mechanisms causing distress. Because subsurface instruments require installation within a borehole and the cost associated with such installations, their use is typically limited to monitoring known features of potential instability or to investigate suspected features. Subsurface deformation measurements monitor the relative movement of zones of rock with respect to each other. Piezometric pressure measurement along zones of potential instability monitor the influence of ground water with respect to stability.

(1) Subsurface deformations. Subsurface deformations within rock slopes are commonly measured with one of two types of downhole instruments, inclinometers, or borehole extensometers.

(a) Inclinometers are installed behind the slope, on flat slopes where drilling access is available, or into the slope and are bottomed in sound, stable rock. Successive

measurements of deflections in the inclinometer are used to determine the depth, magnitude, and rate of lateral movement in the rock mass. While commonly installed in vertical boreholes, inclinometers are available that allow installation in inclined to horizontal boreholes. Because successive deflection measurements can be made at small intervals, the device is ideally suited to precisely locate and define as well as monitor zones of instability. Detailed descriptions of inclinometers can be found in EM 1110-2-1908 (Part 2), "Instrumentation of Earth and Rock Fill Dams" and Dunnicliff (1988).

(b) Borehole extensometers are often placed into the face of a cut or slope to help in determining the zones behind the face which are moving. When a deep cut is being made, extensometers may be installed in the walls as the excavation progresses to monitor the response of the slope to an increasing excavation depth. Multiposition borehole extensometers (MPBX), rod or wire, are able to monitor relative movement of a number of different zones at varying distances behind the cut face. Such measurements help to determine which zones are potentially critical and rate of movement. MPBX's are particularly helpful in distinguishing between surficial and deep-seated movement. Extensometers may be equipped with switches that automatically close and activate warning devices when a preset movement limit is reached. Unless care is taken to isolate downhole wires or rods, installations at great depths are not always practical due to the difficulty of obtaining a straight borehole. It is necessary to eliminate, as much as possible, the friction effects between the extensometer wire or rod and the borehole wall. Friction effects can introduce large errors which make interpretation of the data impossible. The maximum measurable deformation is relatively small ranging from approximately 0.5 to several inches, but this limit can be extended by resetting the instrument. Extensometers are described in EM 1110-2-1908, Part 2 and Dunnicliff (1988).

(2) Piezometric pressure. Drainage of a cut slope is often necessary to increase its stability by reducing pore-water pressures in the slope. The effectiveness of any drainage measure should be monitored by piezometers. Piezometer data should also be used to determine when maintenance of a drainage system is necessary. Piezometers should be installed during site investigation activities to determine the ground-water system. Preconstruction installation is important not only for design of the project but also to determine if construction will adversely affect nearby ground-water users. Data should be obtained before, during, and after construction so that a cause-affect trend can be determined, if there is one. This information is very important if there are claims that conditions in nearby areas have been changed due to activities at the project. Piezometers are discussed in Section III, of this manual.

(3) Anchor loads. When the instruments discussed above indicate that remedial measures such as rockbolts are necessary to stabilize a slope, then these same instruments are used to monitor the effectiveness of the remedial measures. The actual load or tension acting on a rockbolt is monitored with a load cell. This information is to assure that bolts are acting as designed and that the maximum load on the bolt is not exceeded. A representative number of bolts in a system are usually monitored. The types of load cells include the hydraulic, mechanical, strain gaged, vibrating wire, and photoelastic. The strain-gaged load cell is the type most often used to monitor rockbolt systems. Load cells are described in the Rock Testing Handbook as well as Dunnicliff (1988).

10-8. Foundation/Structure Instrumentation

As in the case of cut slopes, foundations and structures such as dams, lock walls, and retaining structures may require a large number and variety of instruments. These instruments are frequently similar or the same as those required for slope monitoring and are divided into three general categories dependent upon what observation is being measured. The three categories include deformation measurements, piezometric pressure measurements, and load/stress measurements.

a. Deformation measurements. Deformations of foundations and structures are generally observed as apparent translation, rotation, or settlement/heave. Apparent deformations may actually be the result of a combination of the above deformation modes.

(1) Translation. Translation deformations caused by foundation/structure interactions are generally apparent as sliding along planes of weakness. It is essential to define the planes along which translation occurs and evaluate the severity of the problem at an early stage. Translation measurements of foundations and structures are generally monitored with subsurface techniques discussed under cut slope instrumentation.

(2) Rotation. A tiltmeter may be used to determine the rate, direction, and magnitude of angular deformation which a rock mass, a structure, or a particular block of rock is undergoing. A tiltmeter, unlike an inclinometer, measures only at a discrete, accessible point. The device may be permanently buried with a remote readout or may

be installed directly on the rock or structure surface. If there is weathered rock at the surface, the device may be mounted on a monument which is founded in or on intact rock. The tiltmeter consists of a reference plate, which is attached to the surface that is being monitored, and a sensing device. A portable sensing device may be installed on the reference plate for each reading or a permanent, waterproof housing containing the sensing device may be installed directly on the surface to be monitored. In the second case, readings may be made from a remote readout station. Tiltmeters may also be installed directly on a structure. Tiltmeters are described in more detail in the Rock Testing Handbook and Dunnicliff (1988).

(3) Settlement/Heave. Settlement refers to compression of the foundation material whereas heave refers to expansion. Mechanisms that cause settlement are discussed in Chapter 5. Mechanisms which cause heave were also briefly discussed in Chapter 5, but are discussed in greater detail in Chapter 12.

(a) Settlement of a foundation beneath a structure may be determined by repeated surveying of the elevation of a settlement gage monument installed directly on the foundation and protected from frost and vandalism. Points on the structure itself may likewise be surveyed to determine settlement, especially if direct access to the foundation is not possible. Settlement indicators may also be used to measure settlement. Settlement indicators are capable of measuring single or multiple points and operate on the same principle as a manometer. In areas beneath buildings or other areas where direct access to the instrument is not available, a remotely read instrument may be used as described by Hanna (1973). The instrument is installed in the foundation before the structure is built. The elevation of the measuring point is calculated using the elevation of the readout point and a pressure reading at the measurement point. The original elevation of the measuring point must be determined for comparisons to later readings.

(b) The floor of an excavation may require monitoring for heave or rebound. Heave is not common in all rock or foundation conditions. Heave measurements give valuable information for use in design of other structures in similar rock masses and conditions. These measurements are also important to correlate performance with design assumptions, especially when the foundation is to support precise industrial or scientific equipment where little departure from the design criteria can be tolerated. Heave points are the most common technique used to measure rebound during excavation. Heave points usually consist of an anchor point that is placed in a borehole at or below the expected elevation at the bottom of the excavation. The elevation of the anchor is determined. The drill hole is filled with a bentonite slurry which contains a dye to aid in relocating the instrument hole during construction. As excavation proceeds, a probe of known length is lowered to the top of the anchor point and the elevation of the anchor point is determined by optical leveling. An alternative method uses a linear potentiometer as the sensing element in the borehole. This type of settlement gage is described by Hanna (1973). Settlement/heave gages are also described in EM 1110-2-1908 (Part 2) and Dunnicliff (1988). The method used for anchoring the reference point to the rock and protection of the instrument during construction are important considerations.

b. Piezometer pressure measurements. As in rock slopes, piezometers are often installed during site investigations and monitored to determine preconstruction conditions. A thorough understanding of the preconstruction conditions is very important not only for determining the effects of such conditions, especially seasonal variations, on the construction and operation of the structure but also for determining the effects of the structure on the ground-water flow system. Dewatering activities, construction of ground-water cutoffs, and reservoir filling may affect local ground-water elevations and flow systems at some distance away from the project possibly producing adverse affects. Once construction begins, piezometers that are not destroyed should continue to be monitored. This information can be used as an indication of how ground-water conditions and pore pressures change due to various construction activities such as removal of overburden or the added weight of the structure. Additional piezometers are installed when the structure is finished to monitor the performance of cutoffs and drainage systems as well as to measure pressures in the foundation underneath a structure or in abutments. The flow rate through the drainage system should be measured as another method of monitoring its performance. Unexplained changes in seepage rates may warn of a serious problem even before it is reflected by piezometer or other instrumentation data. Calibrated weirs or simply a stopwatch and calibrated container for lower flows are commonly used to measure drain flows. Other critical areas should also be instrumented as determined during design. Piezometers are described in more detail in Section III.

c. Load/stress measurements. Instrumentation is frequently required to check design assumptions relating to stress distributions caused by rock/structure interactions as well as to monitor zones of potential distress.

Measurements of stress change in a foundation are made with earth pressure cells which may be installed at the interface of the structure and the rock or in a machined slot within the rock mass. Three commonly used pressure cells, to include vibrating wire, hydraulic (Gloetzl) and WES (similar to Carlson stress meter) type cells, are discussed in EM 1110-2-4300. It is necessary to install a piezometer near a pressure cell to isolate earth pressure changes from pore-water pressure changes. Pressure cells must be installed carefully to eliminate error caused by small localized stress concentrations.

d. Combined measurements. As discussed in Chapter 5, settlement or heave frequently is not uniformly distributed across the foundation. In such cases it may be necessary to monitor the effects of both settlement/heave and structural rotation. Instruments capable of monitoring these combined effects include plumb lines, inverted plumb lines and optical plummets. These devices are thoroughly discussed in EM 1110-2-4300.

Section III
Types of Instruments and Limitations

10-9. General

Section II discussed the general application of a number of different types of instruments commonly used to monitor the performance of cut slopes and foundation rock/structure interactions. References were given that provided detailed descriptions, installation procedures, and limitations as well as advantages and disadvantages of various devices. This section will address two specific types of instruments, piezometers and ground motion/vibration monitoring devices. Piezometers have been mentioned previously but will be covered in greater detail here. Ground motion devices, considered to be location/site specific devices, will be briefly discussed in this section.

10-10. Piezometers

Piezometers are used to measure pore-water pressures and water levels in the natural ground, foundations, embankments, and slopes. Piezometers are also used to monitor the performance of seepage control measures and drainage systems and to monitor the affect of construction and operation of the project on the ground-water system in the vicinity of the project. There are three basic types of piezometers: open-system (open standpipe), closed-system (hydraulic), and diaphragm (pneumatic and electrical, e.g., vibrating wire). The operation, installation, and construction of these piezometers are covered in detail in EM 1110-2-1908. The basic criteria for selecting piezometer types are reliability, simplicity, ruggedness, and life expectancy. Other considerations are sensitivity, ease of installation, cost, and the capability of being monitored from a remote observation point. Sometimes two or more types of piezometers may be required to obtain the most meaningful information at a particular site. One of the most important factors to be considered is the impact of hydrostatic time lag on the intended use of the piezometer data. Table 10-2 compares the different types of piezometers.

a. Open-system piezometers. Open-system piezometers are the simplest types of piezometers but they are also subject to the greatest hydrostatic time lag. They are best used in areas where slow changes in pore-water pressure are expected and the permeability is greater than 10^{-5} cm/sec. If rapid pore water pressure changes are expected, then open-system piezometers should only be used if the permeability is greater than 10^{-3} cm/sec (EM 1110-2-1908, Part 1 of 2).

b. Closed-system piezometers. The rate of pore-water pressure changes has little effect on the measurements obtained with this type of piezometer. This type is commonly used to measure pore pressures during construction of embankments. The readout can be directed to a central location so that there is little interference with construction. However, the device must be checked often for leakage and the presence of air. Open-system piezometers should be installed near key closed-system piezometers to provide a check on the operation of the closed-system piezometer.

c. Diaphragm piezometers. Diaphragm piezometers can be used in the same situations as open and closed system piezometers. They are very sensitive to pore-water pressure changes and the elevation difference between the piezometer tip and the readout point is not a limiting factor. The electrical diaphragm piezometer is complex and may be subject to instrument "zero" drift after calibration and installation, short circuits in the lead cable, stretch and temperature effects in long lead cables, and stray electrical currents.

10-11. Ground Motions/Vibrations

Ground motions/vibrations which can affect a rock foundation may be caused by earthquakes or blasting. Controlled blasting techniques, as discussed in Chapter 11, are used to minimize damage to foundations and adjacent

Table 10-2
Comparison of Piezometer Types

Basic Type	Relative Volume Demand	Readout Equipment	Advantages	Disadvantages
Open-System (standpipe)	High	Water Level Finder	Simple; comparatively inexpensive; generally not subject to freezing; relatively long life; fairly easy to install; long history of effective operation.	Long time lag in most rock types; cannot measure negative pore pressure; cannot be used in areas subject to inundation unless offset standpipe used; must be guarded during construction; no central observation station is possible; requires sounding probe. Must be straight; difficulties possible in small diameter tubes if water levels significantly below 100 feet, or dip less than 45 degrees.
Closed-System (hydraulic)	Medium to low	Usually Bourdon gauge or manometer	Small time lag; can measure negative pore pressures; can be used in areas subject to inundation; comparatively little interference with construction; can be read at central observation stations.	Observation station must be protected against freezing; fairly difficult to install; fairly expensive compared to open systems; sometimes difficult to maintain an air-free system; most types are fragile; some types have limited service behavior records; requires readout location not significantly above lowest water level.
Diaphragm	Low to negligible	Specialized pressure transmitter or electronic readout	Simple to operate; elevation of observation station is independent of elevation of piezometer tip no protection against freezing required; no de-airing required; very small time lag.	Limited performance data, some unsatisfactory experience; some makes are expensive and require expensive readout devices; fragile and requires careful handling during installation.
			Pneumatic. Electrical source not required; tip and readout devices are less expensive than for electrical diaphragm types.	Often difficult to detect when escape of gas starts; negative pressures cannot be measured; condensation of moisture occurs in cell unless dry gas is used; requires careful application of gas pressure during observation to avoid damage to cell.
			Electrical. Negative pressures can be measured; ideal for remote monitoring.	Devices subject to full and partial shortcircuits and repairs to conductors introduce errors; some makes require temperature compensation and have problems with zero drift to strain gages; resistance and stray currents in long conductors are a problem in some makes; zero drift possible.

Note:
1. Modified from Pit Slope Manual, Chapter 4, 1977 and EM 1110-2-1908 (Part 1).

structures caused by blasting. Seismographs should be used to monitor the levels of vibration actually being produced. Seismograph records (seismograms) are also used to provide a record of vibrations to assure maximum levels are not exceeded which could cause damage to adjacent structures. Seismograph is a general term which covers all types of seismic instruments that produce a permanent record of earth motion. The three main types of seismographs measure particle displacement, velocity, and acceleration. The instruments used in different applications are discussed below.

a. Earthquakes. Measurement of earthquake motion assists in damage assessment after a significant earthquake and is necessary for improving the design of structures, especially dams, to better resist earthquakes. Guidance is given in EM 1110-2-1908 for determining which structures require instrumentation. The strong motion

accelerograph and peak recording accelerograph are the principal instruments used to record earthquake motions on engineering projects such as dams. The accelerograph measures particle acceleration in any direction or directions desired. The strong motion instruments generally record seismic motion between 0.01 g and 1.0 g. They are triggered by the minimum level of motion and record continuously during any motion above a preset minimum level and for a short time after motion ceases. The peak accelerograph records only the high amplitudes of the acceleration and does not make a continuous recording. This low cost instrument is used only to supplement data from other accelerographs. One or two strong motion accelerographs may be located on a project and several peak accelerographs may be located in other areas to obtain an idea of how the acceleration differs across the site. EM 1110-2-1908 provides additional discussions.

b. Blasting. As discussed in Chapter 11, construction blasting should be controlled in order to reduce damage by ground vibrations to the foundation being excavated and to nearby structures. Seismographs are used to monitor the ground vibrations caused by blasting. The peak particle velocity is normally used as an indication of potential damage, therefore, a velocity seismograph is normally used in engineering applications. The particle velocity can be inferred from the information obtained by other types of seismographs but it is preferred to measure it directly so that an immediate record is available without extensive processing. EM 1110-2-3800, the *Blaster's Handbook* (Dupont de Nemours and Company 1977), and Dowding (1985) provide additional instrument descriptions.

10-12. Limitations

There are certain requirements by which all types of field instrumentation should be evaluated. These include the range, sensitivity, repeatability, accuracy, and survivability of the instrument. The range must be adequate to measure the expected changes but not so great that sensitivity is lost. It is not always possible to accurately predict the magnitude of loads and deformations to be expected before construction. The most important of these factors may be repeatability because this factor determines the quality of the data. The sensitivity required will vary with the application. Good sensitivity is required for early detection of hazards but may mean a reduction in the range and stability of the instrument. If an instrument with too narrow a range is chosen, all the necessary data may not be obtained. If an instrument with too large a range is chosen, then it may not be sensitive enough. Accuracy is difficult to define and to demonstrate. The anisotropy of a parameter must be predictable if the accuracy is to be determined. Calibration, consistency, and repeatability are also used in determining accuracy. The instrument chosen for a particular application must also be able to survive the often severe conditions under which it will be used. Cost should also be considered and the least expensive way of obtaining good quality information should be used. Table 10-3 provides a summary of some of the major limitations of the various types of instrumentation that have been discussed. Ranges and sensitivities for different instrument types may vary between manufacturers and may change rapidly due to research and development and so are not listed in this table. Many of the instruments are also easily modified by a qualified laboratory to meet the requirements of a particular job.

Section IV
Data Interpretation and Evaluation

10-13. Reading Frequency

The frequency at which instrument readings are taken should be based on many factors and will vary by project, instrument type, availability of government personnel to take readings, and location and may even vary through time. The availability of government personnel to take the readings should be determined during the preparation of plans and specifications. If government personnel will not be available, provisions should be made to have this task performed by the construction contractor or by an A-E contractor. Some of the factors which should be evaluated include outside influences such as construction activities, environmental factors (rainfall events, etc.), the complexity of the geology, rate of ground movements, etc. Several sets of readings should be taken initially to establish a baseline against which other readings are to be evaluated. Daily or even more frequent readings may be necessary during certain construction activities, such as fill placement or blasting. The rate of change of the condition which is being monitored may vary over time, dictating a change in the established frequency at which readings are taken. For example, an unstable slope may move slowly at first, requiring infrequent readings on a regular basis until a near failure condition is reached, at which time readings would have to be taken much more frequently. Readings of different types of instruments should be made at the same time. Concurrent readings enables the interpreter to take into account all the factors which might impact individual readings of specific parameters. For example, an increase in pore water pressure might coincide with increased slope movement. Standard forms should be used to record data when available, or if

Table 10-3
Limitations of Rock Instrumentation

Instrument	Measured Parameter	Limitations
Inclinometer	Deformation	Life may be limited in hard rock due to sharp edges. Significant drilling costs.
Tiltmeter	Deformation	Measures one, near-surface discrete point. Subject to damage during construction. Difficult to detect spurious data. Must be protected from the environment. Subject to errors caused by bonding material.
Extensometers	Deformation	
Bar Single Point		Does not distinguish between deep-seated and surficial movement. Limited accuracy due to sag. Measures only one point. Significant drilling costs, a new drill hole required for each detection point.
Multipoint Rod Multipoint Wire		Limited to approximately 50-foot depth if each rod is not individually cased within the instrument hole. Experienced personnel should install them. May be damaged by borehole debris unless protected. Spring anchors may experience variable spring tension due to rock movement.
Settlement Indicators	Deformation	Hydraulic types require de-aired water. Corrections for temperature and barometric pressure differences are required. Access to drill collar is required for some types.
Heave Point	Deformation	Accuracy is limited by surveying techniques used.
Load Cells	Stress, Load	
Hydraulic		Large size, poor load resolution, temperature sensitivity.
Mechanical		Nonlinear calibration curves.
Strain Gage		Requires waterproofing, long term stable bonding method and periodic recalibration.
Vibrating Wire		Large size, expensive, poor temperature compensation, complicated readout, vulnerable to shock.
Photoelastic		Coarse calibration. Requires access to borehole collar.
Piezometers	Load, Stress	See Table 10-2.
Uplift Cells	Deformation	
Standpipe		Readings may require either of two methods, sounder or pressure gage.
Diaphragm		Susceptible to damage during installation.

not, then forms should be developed for specific instruments. Some forms are shown in EM 1110-2-1908. If possible, data should be reduced in the field and compared with previous readings so that questionable readings can be checked immediately. When large amounts of data must be managed, automatic recording devices that record data as printed output or on magnetic tape for processing by computer should be considered. Too many readings are not necessarily better than too few. An excess of data tends to bog down the interpretation process. A thorough evaluation of the purpose of the instrument program must be used to determine the optimum rate at which readings should be taken, thus assuring that data are obtained when it is needed.

10-14. Automatic Data Acquisition Systems

Automatic data acquisition systems and computer data processing are very popular for obtaining and processing instrumentation data. Computer programs are available for reducing and plotting most types of data. Some of the advantages and disadvantages of these systems are given by Dunnicliff (1988). Use of computer processing can speed much tedious processing but should not replace examination of all of the data by an experienced person.

10-15. Data Presentation

Most types of data are best presented in graphical form. Graphical presentation facilitates the interpretation of relationships and trends in the data. Readings are compared over time and with other instrument readings as well as compared with construction activities and changing environmental conditions. Observed trends should be compared with predicted trends to make an assessment of overall performance. The data should be displayed properly or significant trends may be obscured or may become misleading. A thorough knowledge and understanding of the instrumentation as well as some trial and error is required to successfully accomplish good data presentation. Cookbook interpretation methods are available for some types of data such as that from inclinometers. Cookbook interpretation is discouraged. Every instrument should be carefully and impartially analyzed by experienced personnel, taking all the available information into consideration.

10-16. Data Evaluation

Factors to consider when evaluating instrumentation data include instrument drift, cross sensitivity, calibration, and environmental factors such as temperature and barometric pressure. Instrument drift is the change in instrument readings over time when other factors remain constant. Drift can be caused by temperature fluctuations, power supply instability (weak battery), etc. If drift is not detected, it can lead to erroneous data interpretation. Periodic calibration of instruments when possible, can reduce drift problems. Making repetitious readings also helps to detect and account for drift errors. Field calibration units may be available for some instrument types such as inclinometers. Most instrumentation can be isolated from effects caused by changing environmental conditions through the use of protective housings or relatively inert material. Invar steel is one material that is not greatly affected by temperature change. Where protective measures have not been used, environmental effects must be taken into account or the data may not be useful. Additional information on data processing and presentation may be found in EM 1110-2-1908, Rock Testing Handbook, Hanna (1973) and Dunnicliff (1988).

10-17. Data Use

An instrumentation program can easily fail if the obtained data is never understood and used. A clear understanding of the purpose of the program is necessary for understanding of the data obtained. Some idea of the behavior that is expected of the structure, usually developed during design and adjusted during construction, is necessary in order to evaluate the actual behavior. This predicted behavior is the starting point from which all interpretations are made. With these ideas in mind, instrumentation data should prove to be a helpful tool in clearly understanding and evaluating the behavior of any rock foundation or slope.

Chapter 11
Construction Considerations

11-1. Scope

This chapter provides general guidance for factors to be considered in the construction of foundations and cut slopes excavated in rock masses. The chapter is divided into five sections with general topic areas to include: Excavation; Dewatering and Ground Water Control; Ground Control; Protection of Sensitive Foundation Materials; and Excavation Mapping and Monitoring.

Section I
Excavation

11-2. Information Requirements

The factors that should be considered when determining the applicability of an excavation method fall into two groups. The first group includes the characteristics of the rock mass to be excavated. The more important of these characteristics are hardness or strength of the intact rock, and the degree of fracturing, jointing, bedding, or foliation of the rock mass. This information will normally have been acquired during routine exploration. The second group of factors includes features of the foundation design. These features are the size and shape of the excavation, the tolerances required along the excavation lines, and any restrictions on the time allowed for the excavation to be completed. This second group of factors determines the amount of material to be excavated, the required rate of excavation, the type of finished excavation surface the work must produce, and the amount of working space available.

11-3. Excavation Methods

A number of methods are available for excavating rock. These methods include drill and blast, ripping, sawing, water jets, roadheaders and other mechanical excavation methods.

a. Drill and blast. Drill and blast is the most common method of excavating large volumes of rock. The hardness of some rock types may eliminate most other excavation techniques from consideration for all but the smallest excavations. Blasting methods can be adapted to many variations in site conditions. Drill and blast techniques, materials, and equipment are thoroughly discussed in EM 1110-2-3800 and the Blasters Handbook (Dupont de Nemours and Company 1977). Due to the availability of that manual, the basics of blasting will not be discussed here. The emphasis of this section will be on aspects of design and construction operations that must be considered when blasting is to be used as a foundation excavation method.

(1) Minimizing foundation damage. Blasting may damage and loosen the final rock surfaces at the perimeter and bottom of the excavation. Although this damage cannot be eliminated completely, in most cases it can be limited by using controlled blasting techniques. The more common of these techniques are presplitting, smooth blasting, cushion blasting, and line drilling.

(a) When presplitting, a line of closely spaced holes is drilled and blasted along the excavation line prior to the main blast. This process creates a fracture plane between the holes that dissipates the energy from the main blast and protects the rock beyond the excavation limits from damage.

(b) For the smooth blasting method, the main excavation is completed to within a few feet of the excavation perimeter. A line of perimeter holes is then drilled, loaded with light charges, and fired to remove the remaining rock. This method delivers much less shock and hence less damage to the final excavation surface than presplitting or conventional blasting due to the light perimeter loads and the high degree of relief provided by the open face.

(c) Cushion blasting is basically the same as smooth blasting. However, the hole diameter is substantially greater than the charge diameter. The annulus is either left empty or filled with stemming. The definitions of smooth and cushion blasting are often unclear and should be clearly stated in any blasting specifications.

(d) When using the line drilling method, primary blasting is done to within two to three drill hole rows from the final excavation line. A line of holes is then drilled along the excavation line at a spacing of two to four times their diameter and left unloaded. This creates a plane of weakness to which the main blast can break. This plane also reflects some of the shock from the main blast. The last rows of blast holes for the main blast are drilled at reduced spacing and are lightly loaded. Line drilling is often used to form corners when presplitting is used on the remainder of the excavation.

(e) To minimize damage to the final foundation grade, generally blast holes should not extend below grade. When approaching final grade, the rock should be removed in shallow lifts. Charge weight and hole spacing should also be decreased to prevent damage to the final surface. Any final trimming can be done with light charges, jackhammers, rippers, or other equipment. In certain types of materials, such as hard massive rock, it may be necessary to extend blast holes below final grade to obtain sufficient rock breakage to excavate to final grade. This procedure will normally result in overbreak below the final grade. Prior to placing concrete or some types of embankment material, all loose rock fragments and overbreak must be removed to the contractual standard, usually requiring intense hand labor. The overexcavated areas are then backfilled with appropriate materials.

(2) Adverse effects of blasting. Blasting produces ground vibrations, airblast, and flyrock which affect the area around the site. These effects should be kept to a minimum so that nearby structures and personnel are not damaged, or injured and complaints from local residents are kept to a minimum.

(a) Ground vibration is the cause of most complaints and structural damage. Ground vibration is usually expressed in terms of peak particle velocity, which can be estimated for a certain location using the equation

$$V = H(D/W^{1/2})^{-B} \tag{11-1}$$

where

V = peak particle velocity in one direction, inches per second (ips)

D = distance from blast area to point particle velocity of measurement, ft

W = charge weight per delay, lbs

H, B = constants

The constants, H and B, are site-specific and must be determined by conducting test blasts at the site and measuring particle velocities with seismographs at several different distances in different directions. By varying the charge weight for each blast, a log-log plot of peak particle velocity versus scaled distance ($D/W^{1/2}$) may be constructed. The slope of a best fit straight line through the data is equal to the constant B and the value of velocity at a scaled distance of 1 is equal to the constant H. After determining the constants H and B, Equation 11-1 can then be used to estimate the maximum charge weight that can be detonated without causing damage to nearby structures. If test blasts are not conducted at the site to determine the propagation constants, the maximum charge weight may be estimated by assuming a value for the scaled distance. A value of 50 ft/lb$^{1/2}$ is considered a minimum safe scaled distance for a site for which no seismograph information is available. Using this value,

$$D/W^{1/2} = 50 \text{ ft/lb}^{1/2} \tag{11-2}$$

and

$$W = \left(\frac{D}{50}\right)^2 \tag{11-3}$$

where W is the maximum safe charge weight per delay in pounds. The maximum safe peak particle velocity for most residential structures is approximately 2 ips. Ground vibration exceeding this level may result in broken windows, cracked walls or foundations, or other types of damage. Blasts fired with a high degree of confinement, such as presplit blasts, may cause higher particle velocities than those predicted by the vibration equation. This is due to the lack of relief normally provided by a free excavation face.

(b) Airblast, or compression waves travelling through air, may sometimes damage nearby structures. Noise is that portion of the airblast spectrum having wave frequencies of 20-20,000 Hz. Atmospheric overpressure is caused by the compression wave front. This overpressure may be measured with microphones or piezoelectric pressure gauges. An overpressure of 1 psi will break most windows and may crack plaster. Well-mounted windows are generally safe at overpressures of 0.1 psi, and it is recommended that overpressures at any structure not exceed this level. Airblast is increased by exposed detonating cord, lack of sufficient stemming in blast holes, insufficient burden, heavy low-level cloud cover, high winds, and atmospheric temperature inversions. All of these conditions should be avoided during blasting. Temperature inversions are most common from 1 hour before sunset to 2 hours after sunrise. Blasting should be avoided during these hours if airblast is a concern.

(c) Flyrock is usually caused by loading holes near the excavation face with too heavy a charge or by loading explosives too close to the top of the holes. These conditions should be avoided at all times. Flyrock may also be controlled with blast mats. These are large woven mats

of wire or rope which are laid over the blast holes or on the face to contain flying debris. Blast mats should be used when blasting very close to existing structures. Extreme caution must be used when placing blast mats to prevent damage to exposed blasting circuits. An alternative to a blasting mat is to place a layer of soil a few feet thick over the blast area prior to blasting to contain the flyrock.

(d) Complaints or claims of damage from nearby residents may be reduced by designing blasts to minimize the adverse effects on the surrounding area as much as possible while still maintaining an economic blasting program. To aid in the design of the production blast, test blasts should be conducted and closely monitored to develop attenuation constants for the site. The test blasts should be conducted at several loading factors in an area away from the production blast area or at least away from critical areas of the excavation. However, even with careful blast design, some claims and complaints will most likely occur. People may become alarmed or claim damage when vibration and airblast levels are well below the damage threshold. There are several steps that may be taken to protect against fraudulent or mistaken damage claims. The most basic step is to maintain accurate records of every blast. The blasting contractor is required to submit a detailed blast plan far enough in advance of each shot to allow review by the Government inspector. The blast plan should give all the details of the blast design. After each blast, the contractor should submit a blast report giving the details of the actual blast layout, loading, results, and all other pertinent data. A blast plan and report are normally required on Corps projects. The ground vibrations and airblast from each blast may also be recorded at the nearest structures in several different directions. The seismograph records can be used in the event of a claim to determine if ground vibrations may have reached potentially damaging levels. It is also good practice to record all blasts on videotape. A video-taped record can be helpful in solving various problems with the blasting operations. These monitoring records should be kept, along with the blast plans and records, as a record of the conditions and results of each blast.

(e) A further precaution to be taken to protect against damage claims is to require that the contractor perform a preblasting survey of structures near the blasting area. The purpose of the survey is to determine the condition of nearby structures prior to blasting. The survey should include recording all cracks in plaster, windows, and foundations and photographing the buildings inside and out. The preblast survey might also include basic water quality analyses from any wells in the area. It should also be determined during the survey if there is any sensitive or delicate equipment in nearby buildings that may limit the acceptable peak particle velocity to a value less than the normal 2 ips. This survey should be done at no cost to the property owners. If any property owner refuses to allow his property to be inspected, he should be asked to sign a statement simply stating that he declined the service. The results of the survey will help in determining if damage was pre-existing or is blast-related. The scope of the test blasting, monitoring, and preblast survey will be dependent upon the size and duration of the production blasting and the anticipated sensitivity of the area as determined by the population density and other social and environmental factors.

(f) The key to blasting safety is experienced, safety-conscious personnel. All field personnel directly involved with a blasting operation must be thoroughly familiar with the safety rules and regulations governing the use of explosives. Information and rules on blasting safety are available from explosives manufacturers or the Institute of Makers of Explosives. Safety regulations that apply to Corps of Engineers projects are stated in EM-385-1-1, Safety and Health Requirements, Section 25. These regulations shall be strictly adhered to under all circumstances. The contractor should be required to conduct operations in compliance with all safety regulations. Any unsafe practices must be immediately reported and corrected to avoid accidents.

b. Ripping. Ripping is a means of loosening rock so it may be excavated with loaders, dozers, or scrapers. It involves the use of one or more long narrow teeth which are mounted behind a crawler tractor. Downward pressure is exerted by the tractor and the teeth are pulled through the rock. In addition to standard rippers, impact rippers have been developed in recent years that are capable of breaking relatively strong rock.

(1) Factors influencing rippability. The rock's susceptibility to ripping is related to the rock structure and hardness. The rock structure, in the form of joints, fractures, bedding, faulting, or other discontinuities, determines to a large degree the rippability of the rock mass. These discontinuities represent planes of weakness along which the rock may separate. Rock with closely spaced, continuous, near horizontal fractures is much more easily ripped than rock with widely spaced, discontinuous, high angle fractures. Rock hardness influences the rippability by determining the amount of force that must be exerted by a ripper tooth to fracture the intact rock. Rock type, fabric, and weathering can be related to the rippability of a rock mass because of the influence they have on the

rock structure and hardness. Sedimentary rocks are generally easiest to rip because of their laminated structure. Igneous rock are generally difficult to rip because they are usually hard and lack well-developed lamination. Any weathering that takes place reduces the hardness of the rock and creates additional fractures, making the rock easier to rip. Due to its lesser degree of homogeneity, rock with a coarse grained fabric is generally weaker than fine grained rock. Because of this, coarse grained rock is usually easier to excavate by ripping than finer grained rock types.

(2) Rippability indicators. Seismic wave velocity is often used as an indicator of the rippability of a rock mass. The seismic wave velocity is dependent on the rock density or hardness and the degree of fracturing. Hard, intact rock has a higher seismic velocity than softer, fractured rock. Therefore, rocks with lower seismic velocities are generally more easily ripped than those with higher seismic velocities. The seismic wave velocity may be measured using a refraction seismograph and performing a seismic survey of the excavation site. To determine the rippability of the rock, the seismic wave velocity must then be compared with the seismic wave velocities of similar materials in which ripper performance has been demonstrated. Tractor manufacturers have published charts showing, for a particular size tractor and specific ripper configuration, the degree of rippability for different rock types with varying seismic velocities. The rippability of a rock mass may also be assessed by using a rock mass rating system developed by Weaver (1975). Using this system, various rock mass parameters are assigned numerical ratings. The numerical values are then added together to give a rippability rating. Lower ratings indicate easier ripping. Using tractor manufacturer's charts, this rating can be correlated to production rate for various tractor sizes.

(3) Contract considerations. It should never be stated in contract specifications or other legal documents that a rock is rippable or inability to rip designates a new pay item without specifying the tractor size, ripper configurations and cubic yards loosened per hour (for pay purposes) for which the determination of rippability was made. Rock that may be rippable using a very large tractor may not be rippable using smaller equipment. Not including this qualifying information may lead to claims by a contractor who, after finding he is unable to rip the rock with the size equipment he has available, claims the contract documents are misleading or incorrect.

(4) Other considerations. Ripping may be used to remove large volumes of rock in areas large enough to permit equipment access. However, ripping produces very poorly sorted muck with many large blocks of rock. Muck from ripping may require further breaking or crushing to make it suitable for use as fill or riprap.

c. Sawing. Sawing is not a common practice, although it is sometimes used as a way of trimming an excavation in soft rock to final grade. Saws may also be used to cut a slot along an excavation line prior to blasting or ripping as an alternative to line drilling. One of the advantages of sawing is that it produces a very smooth excavation face with minimal disturbance to the remaining rock. It also gives very precise control of the position of the final excavation face and may be used to finish fairly complex excavation shapes. Coal saws have been used for sawing soft rocks. Concrete saws may be used for very small scale work in harder material.

d. Water jets. High pressure water jets are beginning to find uses as excavation tools in the construction industry. Water jets cut rock through erosion and by inducing high internal pore pressures which fail the rock in tension. Water jets may range from large water canon to small hand-held guns. Extremely hard rock may be cut with water jets. However, the pressures required to cut hard rock are extremely high. Optimum pressures for cutting granite may be as high as 50,000 psi. Water jets may be used for cutting slots, drilling holes, trimming to neat excavation lines, cleaning loose material from an excavated surface. Drill holes may also be slotted or belled. Water jets may not be suitable for use in formations which are extremely sensitive to changes in moisture.

e. Roadheaders. Roadheaders, which are often used in underground excavation, may also be used for final trimming of surface excavations. Roadheaders can rapidly and accurately excavate rock with little disturbance to the remaining rock mass. However, due to power and thrust limitations, their use is limited to rock with a unconfined compressive strength less than approximately 12,000 psi. Large machines may have very high electrical power requirements. Cutting capabilities, length of reach, and power requirements vary widely between models and manufacturers.

f. Other mechanical excavation methods. Various types of mechanical impactors or borehole devices are sometimes used in rock excavation. Mechanical impactors may include hand-operated jackhammers, tractor-mounted rock breakers, or boom-mounted hydraulic impact hammers. These all use chisel or conical points that are driven into the rock by falling weights or by hydraulic or pneumatic hammers. Wedges or hydraulic

borehole jacks may be driven or expanded in boreholes to split the rock. Chemicals have been developed which are placed in boreholes much like explosives and, through rapid crystal growth, expand and fracture the rock. Wedges, borehole jacks, or expanding chemicals may provide alternative means of excavation in areas where the vibration and noise associated with blasting cannot be tolerated because of nearby structures or sensitive equipment. Because of their generally low production rates, these alternative methods are normally used only on a limited basis, where excavation quantities are small or for breaking up large pieces of muck resulting from blasting or ripping. Crane-mounted drop balls are also often used for secondary muck breakage. Jackhammers may be used in confined areas where there is not sufficient room for most equipment to operate.

11-4. Effects of Discontinuities on Excavation

a. Overbreak. The amount of overbreak, or rock breakage beyond intended excavation lines, is strongly affected by the number, orientation, and character of the discontinuities intersecting the faces of the excavation. Discontinuities represent preferred failure planes within the rock mass. During excavation the rock will tend to break along these planes. In rock with medium to closely spaced joints that intersect the excavation face, overbreak will most likely occur and will produce a blocky excavation surface. If joints run roughly parallel to the excavation face, overbreak may occur as slabbing or spalling. Worsey (1981) found that if a major joint set intersected the excavation face at an angle less than 15 degrees, presplit blasting had little or no beneficial effect on the slope configuration. When blasting, overbreak will also be more severe at the corners of an excavation. Overbreak increases construction costs by increasing muck quantities and backfill or concrete quantities. Because of this, the excavation should be planned and carried out in a way that limits the amount of overbreak. Special measures may be required in areas where overbreak is likely to be more severe because of geologic conditions or excavation geometry. These measures may include controlled blasting techniques or changes in the shape of the excavation.

b. Treatment of discontinuities. Sometimes, open discontinuities must be treated to strengthen the foundation or prevent underseepage. Open discontinuities encountered in bore holes below the depth of excavation may be pressure grouted. Open joints and fractures, solution cavities, faults, unbackfilled exploratory holes, or isolated areas of weathered or otherwise unacceptable rock may be encountered during the excavation process. These features must be cleaned out and backfilled. When these features are too small to allow access by heavy equipment normally used for excavation, all work must be done by hand. This process is referred to as dental treatment. Any weathered or broken rock present in the openings is removed with shovels, hand tools, or water jets. The rock on the sides of the opening should be cleaned to provide a good bond with the concrete backfill. Concrete is then placed in the opening, usually by hand.

Section II
Dewatering and Ground Water Control

11-5. Purposes

Dewatering of excavations in rock is performed to provide dry working conditions for men and equipment and to increase the stability of the excavation or structures. Most excavations that are left open to precipitation or that extend below ground water will require some form of dewatering or ground-water control. Evaluation of the potential need for dewatering should always be included in the design of a structure. Construction contract documents should point out any known potential dewatering problems by the field investigation work.

11-6. Planning Considerations

The complexity of dewatering systems varies widely. Small shallow excavations above ground water may require only ditches to divert surface runoff, or no control at all if precipitation and surface runoff will not cause significant construction delays. Extensive dewatering systems utilizing several water control methods may be required for larger deeper excavations where inflow rates are higher and the effects of surface and ground-water intrusion are more severe. It must be determined what ground-water conditions must be maintained during the various stages of the construction of the project. The dewatering system must then be designed to establish and maintain those conditions effectively and economically. The size and depth of the excavation, the design and functions of the planned structure, and the project construction and operating schedule must all be considered when evaluating dewatering needs and methods. The dewatering methods must also be compatible with the proposed excavation and ground support systems. The dewatering system should not present obstacles to excavation equipment or interfere with the installation or operation of the ground support systems. The rock mass permeability and existing ground-water conditions must be determined to evaluate the need for, or adequately design, a ground-water control system. The presence and nature

of fracture or joint-filling material and the hardness or erodibility of the rock should also be determined to assess the potential for increasing flows during dewatering due to the enlargement of seepage paths by erosion.

11-7. Dewatering Methods

Dewatering refers to the control of both surface runoff and subsurface ground water for the purpose of enhancing construction activities or for improving stability.

a. Surface water control. Runoff and other surface waters should be prevented from entering the excavation by properly grading the site. Ditches and dikes may be constructed to intercept runoff and other surface water and direct it away from the work area. Ponding of water on the site should be prevented. Ponded water may infiltrate and act as a recharge source for ground-water seepage into the excavation.

b. Ground-water control. Ground water may be controlled by a number of different methods. The more commonly used methods include open pumping, horizontal drains, drainage galleries, wells, and cutoffs.

(1) Open pumping. When dewatering is accomplished with the open pumping method, groundwater is allowed to enter the excavation. The water is diverted to a convenient sump area where it is collected and pumped out. Collector ditches or berms constructed inside the excavation perimeter divert the water to sumps. Pumps are placed in pits or sumps to pump the water out of the excavation. Most large excavations will require some form of open pumping system to deal with precipitation. In hard rock with clean fractures, fairly large groundwater flows can be handled in this manner. However, in soft rock or in rock containing soft joint filling material, water flowing into the excavation may erode the filling material or rock and gradually increase the size of the seepage paths, allowing flows to increase. Other conditions favorable for the use of open pumping are low hydraulic head, slow recharge, stable excavation slopes, large excavations, and open unrestricted work areas. Open-pumping dewatering systems are simple, easily installed, and relatively inexpensive. However, dewatering by open pumping does not allow the site to be drained prior to excavation. This may result in somewhat wetter working conditions during excavation than would be encountered if the rock mass were predrained. Another disadvantage is that the water pressure in low permeability rock masses may not be effectively relieved around the excavation. This method should not be used without supplementary systems if the stability of the excavation is dependent on lowering the piezometric head in the surrounding rock mass. Because the drainage system lies inside the excavation, it may interfere with other construction operations. In some cases, it may be necessary to overexcavate to provide space for the drainage system. If overexcavation is required, the cost of the system may become excessive.

(2) Horizontal drains. Horizontal drains are simply holes drilled into the side of the excavation to intercept high angle fractures within the rock mass. The drain holes are sloped slightly toward the excavation to allow the water to drain from the fractures. The drains empty into ditches and sumps and the water is then pumped from the excavation. This is a very effective and inexpensive way to relieve excess pore pressure in the rock mass behind the excavation sides or behind a permanent structure. The drain holes can be drilled as excavation progresses downward and do not interfere with work or equipment operation after installation. When laying out drain hole locations, the designers must make sure they will not interfere with rockbolts or concrete anchors.

(3) Drainage galleries. Drainage galleries are tunnels excavated within the rock mass outside the main excavation. Drainage galleries normally are oriented parallel to the excavation slope to be drained. Radial drain holes are drilled from the gallery to help collect the water in fractures and carry it into the drainage gallery, where it is then pumped out. Drainage galleries must be large enough to permit access of drilling equipment for drilling the drain holes and future rehabilitation work. This method is effective in removing large quantities of water from the rock mass. Drainage galleries can be constructed prior to the foundation excavation using conventional tunnel construction methods to predrain the rock mass and they may be utilized as a permanent part of the drainage system for a large project. However, they are very expensive to construct and so are only used when water must be removed from a large area for extended periods of time.

(4) Wells. Pumping wells are often used to dewater excavations in rock. Wells can be placed outside the excavation so they do not interfere with construction operations. Wells also allow the rock mass to be predrained so that all excavation work is carried out under dry conditions. Wells are capable of producing large drawdowns over large areas. They are also effective for dewatering low to medium angle fractures that may act as slide planes for excavation slope failures. They will not effectively relieve the pore pressure in rock masses in which the jointing and fracturing is predominantly high

angled. The high angle fractures are not likely to be intersected by the well and so will not be dewatered unless connected to the well by lower angled fractures or permeable zones. The operating cost of a system of pumping wells can be high due to the fact that a pump must operate in each well. Power requirements for a large system can be very high. A backup power source should always be included in the system in the event of failure of the primary power source. Loss of power could result in failure of the entire system.

(5) Cutoffs. Ground-water cutoffs are barriers of low permeability intended to stop or impede the movement of ground water through the rock mass. Cutoffs are usually constructed in the form of walls or curtains.

(a) Grouting is the most common method of constructing a cutoff in rock. A grout curtain is formed by pressure grouting parallel lines of drill holes to seal the fractures in the rock. This creates a solid mass through which ground water cannot flow. However, complete sealing of all fractures is never achieved in grouting. The effectiveness of a grout curtain is difficult to determine until it is in operation. Measurements of changes in grout injection quantities during grouting and pumping tests before and after grouting are normally used to estimate the effectiveness of a grouting operation. Grouting for excavation dewatering can normally be done outside the excavation area and is often used to reduce the amount of water that must be handled by wells or open pumping. It is also used to construct permanent seepage cutoffs in rock foundations of hydraulic structures. Corps of Engineers publications on grouting include EM 1110-2-3506, EM 1110-2-3504, Albritton, Jackson, and Bangert (1984) (TR GL-84-13).

(b) Sheet pile cutoffs may be used in some very soft rocks. However, sheet piling cannot be driven into harder materials. The rock around the sheet pile cutoff may be fractured by the pilings during installation. This will increase the amount of flow around and beneath the cutoff wall and greatly reduce its effectiveness.

(c) Slurry walls may also be used as cutoffs in rock. However, due to the difficulty and expense of excavating a deep narrow trench in rock, slurry walls are usually limited to use in soft rocks that may be excavated with machinery also used in soils.

(d) Recent developments in mechanical rock excavators that permit excavation of deep slots in relatively strong and hard rock have resulted in increased cost-effectiveness of using diaphragm walls as effective cutoff barriers.

(e) Ground freezing may be used to control water flows in areas of brecciated rock, such as fault zones. The use of freezing is generally limited to such soil-like materials. The design, construction, and operation of ground freezing systems should be performed by an engineering firm specializing in this type of work.

Section III
Ground Control

11-8. Stability Through Excavation Planning

During the design or construction planning stages of a project that involve significant cuts in rock, it is necessary to evaluate the stability of the planned excavations. The stability of such excavations is governed by the discontinuities within the rock mass. The occurrence, position, and orientations of the prominent discontinuities at a site should be established during the exploration phase of the project. Using the information and the proposed orientations of the various cut faces to be established, vector analysis or stereonet projections may be used to determine in which parts of the excavation potentially unstable conditions may exist. If serious stability problems are anticipated, it may be possible to change the position or orientation of the structure or excavation slope to increase the stability. However, the position of the structure is usually fixed by other factors. It may not be practical to change either its position or orientation unless the stability problems created by the excavation are so severe that the cost of the necessary stabilizing measures becomes excessive. It may never be possible to delineate all discontinuities and potentially unstable areas before excavation begins. Unexpected problems will likely always be exposed as construction progresses and will have to be dealt with at that time. But performing this relatively simple and inexpensive analysis during design and planning can reduce construction costs. The costs and time delays caused by unexpected stability problems or failures during construction can be extreme. The level of effort involved in determining the stability of the excavation slopes will be governed by the scale of the project and the consequences of a failure. A very detailed stability analysis may be performed for a dam project involving very deep foundation cuts where a large failure would have a serious impact on the economics and safety of the operation. The level of effort for a building with a shallow foundation may only include a surface reconnaissance survey of any exposed rock with minimal subsurface

investigations and then any unstable portions of the excavation may be dealt with during construction.

11-9. Selection of Stabilization Measures

When choosing a stabilization method, it is important that the applicable methods be compared based on their effectiveness and cost. In some cases, it may be permissible to accept the risk of failure and install monitoring equipment to give advance warning of an impending failure. Hoek and Bray (1977) gives a practical example of selecting a stabilization method from several possible alternatives.

11-10. Stabilization Methods

Remedial treatment methods for stabilizing slopes excavated in rock were briefly discussed in Chapter 8. Stabilization methods to include drainage slope configuration, reinforcement, mechanical support and shotcrete are discussed in more detail below.

a. Drainage. The least expensive method of increasing the stability of a slope is usually to drain the ground water from the fractures. This can be done by horizontal drain holes drilled into the face, vertical pumping wells behind the face, or drainage galleries within the slope. In conjunction with drainage of the ground water, surface water should be kept from entering the fractures in the slope. The ground surface behind the crest should be sloped to prevent pooling and reduce infiltration. Diversion ditches may also be constructed to collect runoff and carry it away from the slope. Diversion and collection ditches should be lined if constructed in highly permeable or moisture sensitive materials.

b. Slope configuration. Other stabilization methods involve excavating the slope to a more stable configuration. This can be done by reducing the slope angle or by benching the slope. Benching results in a reduced overall slope angle and the benches also help to protect the work area at the base of the slope from rockfall debris. If the majority of the slope is stable and only isolated blocks are known to be in danger of failing, those blocks may simply be removed to eliminate the problem. The use of controlled blasting techniques may also improve the stability of an excavated slope by providing a smoother slope face and reducing the amount of blast-induced fracturing behind the face.

c. Rock reinforcement. Rock reinforcement may be used to stabilize an excavation without changing the slope configuration and requiring excess excavation or backfill. Rock bolts or untensioned dowels are used to control near surface movements and to support small to medium sized blocks. They may be installed at random locations as they are needed or in a regular pattern where more extensive support is required. Rock anchors or tendons are usually used to control movements of larger rock masses because of their greater length and higher load capacity. One of the advantages of using reinforcement is that the excavation face may be progressively supported as the excavation is deepened. Thus, the height of slope that is left unsupported at any one time is equal to the depth of a single excavation lift or bench. After installation, rock reinforcement is also out of the way of activity in the work area and becomes a permanent part of the foundation. Rock bolts or anchors may also be installed vertically behind the excavation face prior to excavation to prevent sliding along planar discontinuities which will be exposed when the cut face is created. The effects of rock reinforcement are usually determined using limit equilibrium methods of slope analysis. Methods for determining anchorage force and depth are given in Chapter 9 on Anchorage Systems. While the methods discussed in Chapter 9 were primarily developed for calculating anchor forces applicable to gravity structures, the principles involved are also applicable to rock slopes. Additional information may be found in EM 1110-1-2907 (1980) and in the references cited in Chapter 8.

d. Mechanical support and protection methods. Mechanical support methods stabilize a rock mass by using structural members to carry the load of the unstable rock. These methods do not strengthen the rock mass. The most common type of mechanical support for foundation excavations is bracing or shoring. In rock excavations, support usually consists of steel beams placed vertically against the excavation face. In narrow excavations, such as trenches, the vertical soldier beams are held in place by horizontal struts spanning the width of the trench. In wider excavations, the soldier beams are supported by inclined struts anchored at the lower end to the floor of the excavation. Steel or timber lagging may be placed between the soldier beams where additional support is needed. One of the disadvantages of bracing and shoring is that mobility in the working area inside the excavation is hampered by the braces. A common solution to this problem is to tie the soldier beams to the rock face with tensioned rock bolts. This method utilizes the benefits of rock reinforcement while the beams spread the influence of each bolt over a large area. When only small rock falls are expected to occur, it may not be necessary to stabilize the rock. It may only be necessary to protect the work area in the excavation from the falling debris. Wire mesh pinned to the face with short dowels will

prevent loose rock from falling into the excavation. The mesh may be anchored only at its upper edge. In this case, the falling debris rolls downslope beneath the mesh and falls out at the bottom of the slope. Wire mesh may be used in conjunction with rock bolts and anchors or bracing to help protect workers from debris falling between larger supports. Buttresses, gabions, and retaining walls, although commonly used for support of permanent slopes, are not normally used to support temporary foundation excavations.

e. Shotcrete. The application of shotcrete is a very common method of preventing rock falls on cut rock slopes. Shotcrete improves the interlock between blocks on the exposed rock surface. The shotcrete does not carry any load from the rock and so is more a method of reinforcement than of support. Shotcrete may also be applied over wire mesh or with fibers included for added strength and support. Shotcrete is fast and relatively easy to apply and does not interfere with workings near the rock cut. Shotcrete also aids in stabilizing rock cuts by inhibiting weathering and subsequent degradation of the rock. This is discussed further in Section IV on Protection of Sensitive Foundations.

Section IV
Protection of Sensitive Foundation Materials

11-11. General

Some rocks may weather or deteriorate very rapidly when exposed to surface conditions by excavation processes. These processes may cause a considerable decrease in the strength of the near surface materials. The processes most likely to be responsible for such damage are freeze-thaw, moisture loss or gain, or chemical alteration of mineral constituents. To preserve the strength and character of the foundation materials, they must be protected from damaging influences.

11-12. Common Materials Requiring Protection

There are several rock types that, because of their mineralogy or physical structure, must be protected to preserve their integrity as foundation materials.

a. Argillaceous rocks. Shales and other argillaceous rocks may tend to slake very rapidly when their moisture content decreases because of exposure to air. This slaking causes cracking and spalling of the surface, exposing deeper rock to the drying effects of the air. In severe cases, an upper layer of rock may be reduced to a brecciated, soil-like mass.

b. Swelling clays. Joint filling materials of montmorillonitic clays will tend to swell if their moisture content is increased. Swelling of these clays brought about by precipitation and runoff entering the joints may cause spalling or block movement perpendicular to the joints.

c. Chemically susceptible rock. Some rock types contain minerals that may chemically weather at a very rapid rate to a more stable mineral form. The feldspars in some igneous rocks and the chlorite and micas in some schists may rapidly weather to clays when exposed to air and water. This process can produce a layer of clayey, ravelling material over the surface of hard, competent rock.

d. Freeze/thaw. Most rocks are susceptible to some degree to damage from freezing. Water freezing in the pores and fractures of the rock mass may create high stresses if space is not available to accommodate the expansion of the ice. These high stresses may create new fractures or enlarge or propagate existing fractures, resulting in spalling from the exposed face.

11-13. Determination of Protection Requirements

The susceptibility of the foundation materials to rapid deterioration or frost damage should be determined during the exploration phase of a project. If possible, exposures of the materials should be examined and their condition and the length of time they have been exposed noted. If core samples are taken as part of the exploration program, their behavior as they are exposed to surface conditions is a very good indication of the sensitivity of the foundation materials to moisture loss. Samples may also be subjected to freeze-thaw and wet-dry cycles in the laboratory. The behavior of the rock at projects previously constructed in the same materials is often the best source of information available provided the construction process and schedule are similar. In this respect, the project design, construction plan, and construction schedule play important roles in determining the need for foundation protection. These determine the length of time excavated surfaces will be exposed. Climatic conditions during the exposure period will help determine the danger of damage from frost or precipitation.

11-14. Foundation Protection Methods

The first step in preventing damage to sensitive foundation materials is to plan the construction to minimize the length of time the material is exposed. Construction specifications may specify a maximum length of time a surface may be exposed without requiring a protective

coating. Excavation may be stopped before reaching final grade or neat excavation lines if a surface must be left exposed for an extended period of time. This precaution is particularly wise if the material is to be left exposed over winter. The upper material that is damaged by frost or weathering is then removed when excavation is continued to final profiles and the rock can be covered more quickly with structural concrete. It may not be possible to quickly cover the foundation materials with structural concrete. In this case it is necessary to temporarily protect the foundation from deterioration. This can be done by placing a protective coating over the exposed foundation materials.

a. Shotcrete. Sprayed-on concrete, or shotcrete, is becoming perhaps the most common protective coating for sensitive foundation materials. Its popularity is due largely to the familiarity of engineers, inspectors, and construction contractors with its design and application. Shotcrete can be easily and quickly applied to almost any shape or slope surface. If correctly applied, it prevents contact of the rock with air and surface water. If ground water is seeping from the rock, weep holes should be made in the shotcrete to help prevent pressure buildup between the rock and the protective layer. Otherwise, spalling of the shotcrete will most likely occur. The shotcrete may be applied over wire mesh pinned to the rock to improve the strength of the protective layer. When used as a protective coating only, the thickness of the shotcrete will normally be 2-3 inches.

b. Lean concrete or slush grouting. Slush grouting is a general term used to describe the surface application of grout to seal and protect rock surface. The grout used is usually a thin sand cement grout. The mix is spread over the surface with brooms, shovels, and other hand tools and worked into cracks. No forms of any kind are used. Lean concrete may also be specified as a protective cover. It is similar to slush grouting in that it is placed and spread largely by hand. However, the mix has a thicker consistency and a thicker layer is usually applied. Because of the thicker application, some forming may be necessary to prevent lateral spreading. Both methods provide protection against surface water and moisture loss to the air. The use of slush grouting and lean concrete for protection are limited to horizontal surfaces and slopes of less than about 45 degrees due to the thin mixes and lack of forming.

c. Plastic sheeting. Sheets of plastic, such as polyethylene, may be spread over foundation surfaces to prevent seepage of surface moisture into the rock. This may also provide a small degree of protection from moisture loss for a short time. Sheet plastics work best on low to medium angled slopes. The plastic sheets are difficult to secure to steep slopes, and water may stand on horizontal surfaces and penetrate between sheets. The sheets can be conveniently weighted in place with wire mesh.

d. Bituminous coatings. Bituminous or asphaltic sprays may also be used as protective coatings. These sprays commonly consist of asphalt thinned with petroleum distillates. The mixture is heated to reduce its viscosity and is then sprayed onto the rock surface. These coatings are effective as temporary moisture barriers. However, they are not very durable and usually will not remain effective for more than 2 to 3 days.

e. Resin coatings. Various synthetic resins are manufactured for use as protective coatings for rock, concrete, and building stone. These products generally form a low permeability membrane when sprayed on a surface. The membrane protects the rock from air and surface water. Life expectancy, mixes, and materials vary with different manufacturers. These materials require specialized equipment and experienced personnel for application. Resin coatings may need to be removed from rock surfaces prior to placement of structural concrete to assure proper rock/concrete bond. Sources of additional information are limited due to the somewhat limited use of these coatings. Potential suppliers of these materials may include manufacturers of coatings, sealers, or resin grouts.

Section V
Excavation Mapping and Monitoring

11-15. Mapping

Geologic mapping should be an integral part of the construction inspection of a foundation excavation. This mapping should be performed by the project geologist who will prepare the Construction Foundation Report required by ER 1110-2-1801. Thorough construction mapping ensures that the final excavation surfaces are examined and so aids in the discovery of any unanticipated adverse geologic conditions. Mapping also provides a permanent record of the geologic conditions encountered during construction. Appendix B of EM 1110-1-1804, Geotechnical Investigations, and Chapter 3 of this report outline procedures for mapping open excavations.

11-16. Photography

Photographs should be taken of all excavated surfaces and construction operations. As with mapping, photographs should be taken by the person(s) responsible for

preparation of the Construction Foundation Report (ER 1110-2-1801). However, project staffing may be limited such that it may be necessary to require the contractor to take the photographs. All photos must be properly labeled with date, subject, direction of view, vantage point, photographer, and any other pertinent information. Photographs of excavated surfaces should be as unobstructed as possible. Complete photographic coverage of the project is very important. Recently, videotaping has also provided benefits. This should be impressed upon the geologists and engineers responsible for construction mapping and inspection.

11-17. Construction Monitoring

Monitoring of construction procedures and progress should be performed on a regular basis by the designers in accordance with ER 1110-2-112. The schedule of design visits should be included in the Engineering Considerations and Instructions to Field Personnel. Excavation monitoring must be performed as thoroughly and frequently as possible to ensure that complete information is obtained on the as-built condition of the rock foundation. A checklist may be used that allows the inspector to give a brief description of various features of the foundation and the construction activities. An example of such a checklist is given in Appendix B of EM 1110-1-1804.

Chapter 12
Special Topics

12-1. Scope

This chapter provides general guidance in recognizing and treating special conditions which can be encountered in rock foundations that cause construction or operation problems. These conditions are likely to be encountered only within certain regions and within certain rock types, but geotechnical professionals should be aware of the potential problems and methods of treatment. This chapter is divided into three topic areas: karst, pseudokarst, and mines which produce substantial underground cavities; swelling and squeezing rock, much of which may be described as a rock but treated as a soil; and gradational soil-rock contacts, rock weathering, saprolites, and residual soils which make determination, selection, and excavation of suitable bearing elevations difficult.

Section I
Karst, Pseudokarst, and Mines

12-2. Cavities in Rock

A topic of concern in many projects involving rock excavation is whether or not there are undetected cavities below an apparently solid bedrock surface or whether cavities could develop after construction. These cavities may occur naturally in karst or pseudokarst terrains, may be induced by human interference in natural processes, or they may be totally due to man's activities. The term "cavities" is used since it covers all sizes and origins of underground openings of interest in rock excavations.

a. Cavity significance. The presence of cavities has a number of rock engineering implications, including:

(1) Irregular or potentially irregular bedrock topography due to collapse or subsidence and associated unpredictable bearing surface elevations.

(2) Excavation difficulties, with extensive hand-cleaning, grouting, and dental treatment requirements.

(3) Questionable support capacity with a potential for collapse or subsidence over cavities, or settlement of debris piles from prior collapses, all of which may be concealed by an apparently sound bedrock surface.

(4) Ground water flow problems, with requirements for tracing flow paths, or sealing off or diverting flows around or through the project area. Surface water flows may be affected by underground cavities, sometimes by complete diversion to the subsurface.

(5) Contaminants may flow rapidly into open channels, with minimal natural filtration and purification, possibly contaminating local water supplies.

b. Problem rocks.

(1) Most natural and induced cavities develop in soluble rocks, most notably limestone, dolomite, gypsum, and rock salt. Typical karst conditions develop in limestones and dolomites by solution-widening of joints and bedding planes caused by flowing ground water. Eventually, this process develops into a heterogeneous arrangement of cavities with irregular sinkholes occurring where cavity roofs have collapsed. The amount of solution that occurs in limestone and dolomite would be negligible in the lifetime of a typical project. Hence, existing cavities are the major concern.

(2) Gypsum and anhydrite are less common than limestones, but they have the additional concern of solution and collapse or settlement during the useful life of a typical structure. Flow of ground water, particularly to water supply wells, has been known to dissolve gypsum and cause collapse of structures. Rock salt is probably one of the most soluble of common geologic materials, and may be of concern in some areas, particularly along the Gulf of Mexico, the Michigan Basin, and in central Kansas. While natural occurrences of cavities in rock salt are rare, cavities may have been formed by solution mining methods, and collapse or creep has occurred in some of the mined areas.

(3) Pseudokarst terrain is an infrequently encountered form that appears to be classic karst topography, but occurs in a different geologic environment. Cavities and sinkholes can occasionally occur in lava flow tubes, or in poorly cemented sandstones adjacent to river valleys or coastlines. The same basic engineering problems and solutions apply to pseudokarst as to karst topography, but generally on a less severe scale. Care should be taken to avoid attributing surface features to pseudokarst conditions, when true karst conditions in lower rock strata may be the actual cause.

c. Mining activities. Mining is the principle cause of human-induced cavities, and subsidence or collapse over old mines is one of the oldest forms of surface disruption caused by man. Coal with occurrences shown in

Figure 12-1 is probably the most common material extracted by underground mining, although nearly any valuable mineral may have been mined using any scale of mining operation. The mines typically follow beds or ore bodies that are relatively easy to follow using stratigraphic or structural studies. The actual locations of mined cavities may be more difficult to determine. Mines in recent times generally have excellent layout maps available, but older mines may not be well documented. In some cases, small scale prospect operations may be totally obscured until excavations are at an advanced stage.

12-3. Investigations

Cavities are difficult to detect, and are undiscovered until exposed by construction excavations. A combination of detailed preconstruction investigations and construction investigations should be anticipated in potential cavity areas. In this respect, karst topography develops in relatively predictable regions of limestones and dolomites, as shown in Figure 12-2 and Table 12-1. However, the occurrence of cavities on a local scale is more difficult to determine, and many significant cavities can be missed by a typical exploration program. The inability to detect specific cavities also holds true for pseudokarst terrains, rock salt, gypsum, and mine cavities. The Geotechnical Investigations Manual, EM 1110-1-1804, provides guidance on the screening of an area for sinkholes, anhydrites or gypsum layers, caves, and area subsidence.

a. Initial site investigations. Geophysics may be of some use in initial site investigations in locating larger cavities, but may miss smaller ones. Remote sensing using air photos, infrared imagery, and side-looking radar are useful in determining trends of cavities and jointing in an area, as well as determining structural geology features associated with rock salt exposures. Detailed joint strike and dip mapping, in some cases by removing site overburden, may be very useful in predicting the trends of known cavities which follow joints. In some cases,

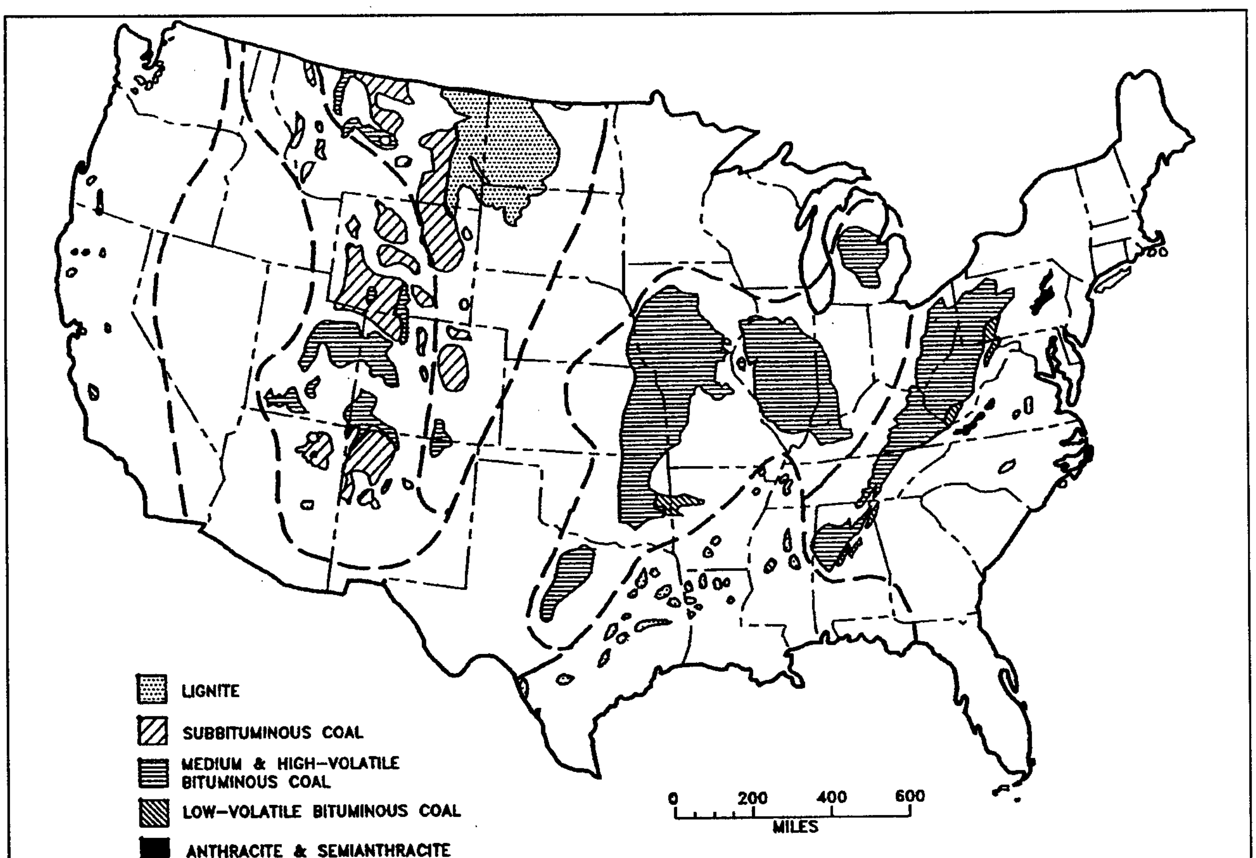


Figure 12-1. Location of coal fields in the United States

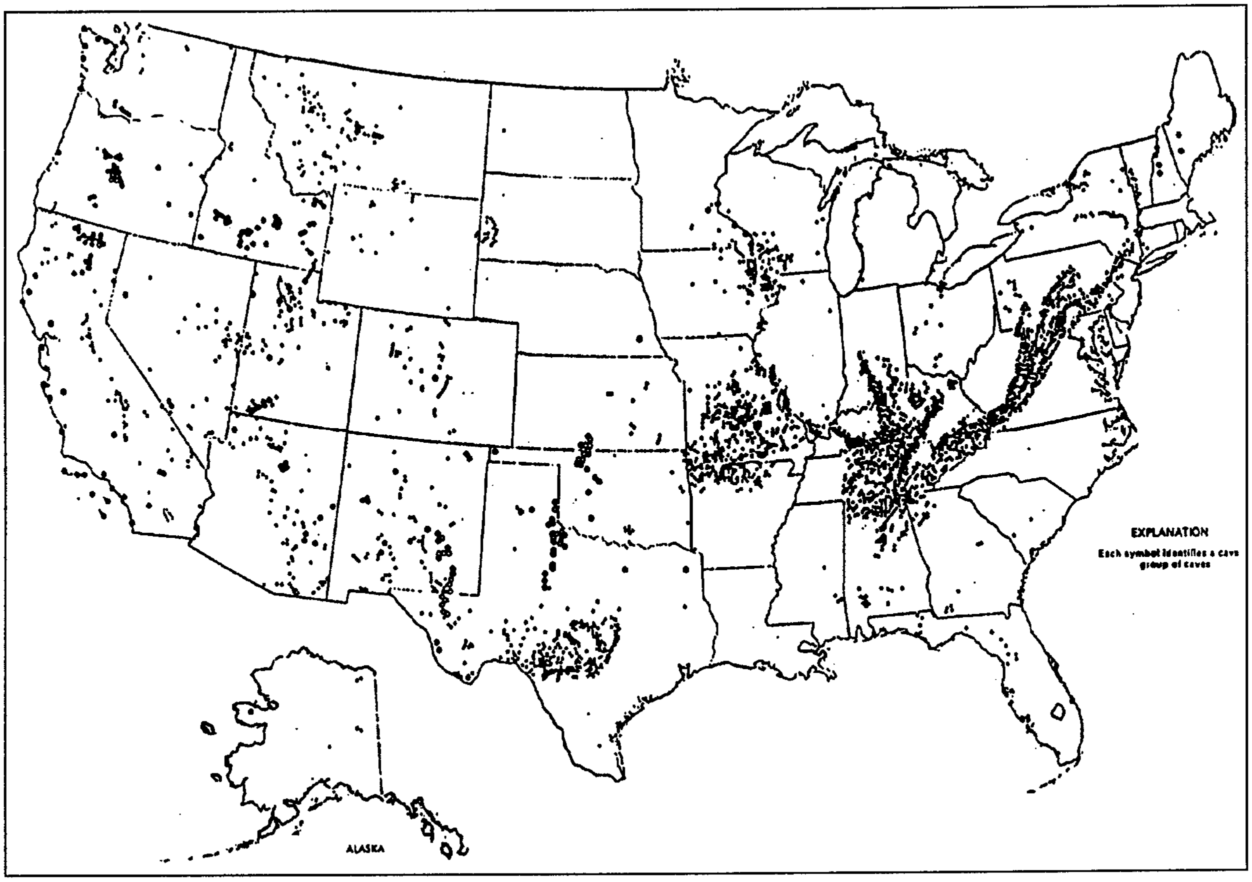


Figure 12-2. Location of cavern areas in the United States

hydrologic testing using piezometers, dye flow tracers, and pump tests may help determine permeabilities and probable flow paths along cavities. In the case of mines, stratigraphic analysis of economic minerals and ore body studies, along with studies of mining company records and Government documents associated with the mine, can help in determining the mine layout. Surveys from inside mines are desirable, but may not be possible due to dangerous conditions. Borehole cameras may be used to determine the size and condition of otherwise inaccessible mines. Table 12-2 shows several exploration and investigation methods which may be of more value in detection of cavities.

b. Cavity detection. Since cavity occurrence is difficult to determine on a local scale, the only practical solution, after initial site studies, is to place a test boring at the location of each significant load-bearing member. Such an undertaking is costly, but represents the only reasonable approach in areas of high concern.

12-4. Alternative Solutions

A number of techniques/methods are available for addressing design and construction problems associated with project sites where cavities are present. The following provides a brief listing of alternative techniques.

a. Avoid the area for load-bearing use if possible.

b. Bridge the cavity by transferring the loads to the cavity sides.

c. Allow for subsidence and potentially severe differential settlements in the design of the foundation and structure.

d. Fill in the cavities to minimize subsidence, prevent catastrophic collapse, and prevent progressive enlargement. Support piers or walls may be used for point supports in larger cavities, or cavities may be filled

Table 12-1
Summary of Major Karst Areas of the United States

Karst Area	Location	Characteristics
Southeastern coastal plain	South Carolina, Georgia	Rolling, dissected plain, shallow dolines, few caves; Tertiary limestone generally covered by thin deposits of sand and silt.
Florida	Florida, southern Georgia	Level to rolling plain: Tertiary, flat-lying limestone; numerous dolines, commonly with ponds; large springs; moderate sized caves, many water filled.
Appalachian	New York, Vermont, south to northern Alabama	Valleys, ridges, and plateau fronts formed south of Palaeozoic limestones, strongly folded in eastern part; numerous large caves, dolines, karst valleys, and deep shafts; extensive areas of karren.
Highland Rim	central Kentucky, Tennessee, northern Georgia	Highly dissected plateau with Carboniferous, flat-lying limestone; numerous large caves, karren, large dolines and uvala.
Lexington-Nashville	north-central Kentucky, central Tennessee, south eastern Indiana	Rolling plain, gently arched; Lower Palaeozoic limestone; a few caves, numerous rounded shallow dolines.
Mammoth Cave-Pennyroyal Plain	west-central, southwestern Kentucky, southern Indiana	Rolling plain and low plateau; flat-lying Carboniferous rocks; numerous dolines, uvala and collapse sinks; very large caves, karren developed locally, complex subterranean drainage, numerous large "disappearing" streams.
Ozarks	southern Missouri, northern Arkansas	Dissected low plateau and plain; broadly arched Lower Palaeozoic limestones and dolomites; numerous moderate-sized caves, dolines, very large springs; similar but less extensive karst in Wisconsin, Iowa, and northern Illinois.
Canadian River	western Oklahoma, northern Texas	Dissected plain, small caves and dolines in Carboniferous gypsum.
Pecos Valley	western Texas, southeastern New Mexico	Moderately dissected low plateau and plains; flat-lying to tilted Upper Palaeozoic limestones with large caves, dolines, and fissures; sparse vegetation; some gypsum karst with dolines.
Edwards Plateau	southwestern Texas	High plateau, flat-lying Cretaceous limestone; deep shafts, moderate-sized caves, dolines; sparse vegetation.
Black Hills	western South Dakota	Highly dissected ridges; folded (domed) Palaeozoic limestone; moderate-sized caves, some karren and dolines.
Kaibab	northern Arizona	Partially dissected plateau, flat-lying Carboniferous limestones; shallow dolines, some with ponds; few moderate-sized caves.
Western mountains	Wyoming, north western Utah, Nevada, western Montana, Idaho, Washington, Oregon, California	Isolated small areas, primarily on tops and flanks of ridges, and some area in valleys; primarily in folded and tilted Palaeozoic and Mesozoic limestone; large caves, some with great vertical extent, in Wyoming, Utah, Montana, and Nevada; small to moderate-sized caves elsewhere; dolines and shafts present; karren developed locally.

Table 12-2
Effectiveness of Cavity Investigation Techniques

		Investigation Method Considered[1]						
Cavity Type	Increased Borings	Geophysics	Remote Sensing	Piezometers	Pump Tests Dye Flow Tests	Discontinuity Analysis	Borehole Cameras	Mine Record Studies
Anhydrite Gypsum	2	1	2	1	1	3	1	2
Karst	5	4	4	2	3	5	5	1
Salt	2	3	3	1	1	1	2	3
Mines	4	4	4	1	1	1	5	5
Lava Tubes	4	2	4	2	2	1	3	1

Note:
1. Ratings: Grade from 1 = not effective to 5 = highly effective

with sand, gravel, and grout. Cement grout can be used to fill large cavities to prevent roof slabs from falling, eliminating a potential progression to sinkholes. Grout also can fill cavities too small for convenient access, thereby reducing permeability and strengthening the rock foundation.

e. Avoid placing structures over gypsum, salt, or anhydrite beds where seeping or flowing water can rapidly remove the supporting rock.

f. Plan for manual cleaning of pinnacled rock surfaces with slush grouting and dental treatment of enlarged joints as shown in Figure 12-3. The exact extent of this work is difficult to predict prior to excavation.

g. Control surface and ground-water flow cautiously. Lowering of the water table has induced collapses and the formation of new sinkholes in previously unexpected areas. Surface drainage in most karst areas is poorly developed, since most drainage has been to the subsurface.

Section II
Swelling and Squeezing Rock

12-5. General

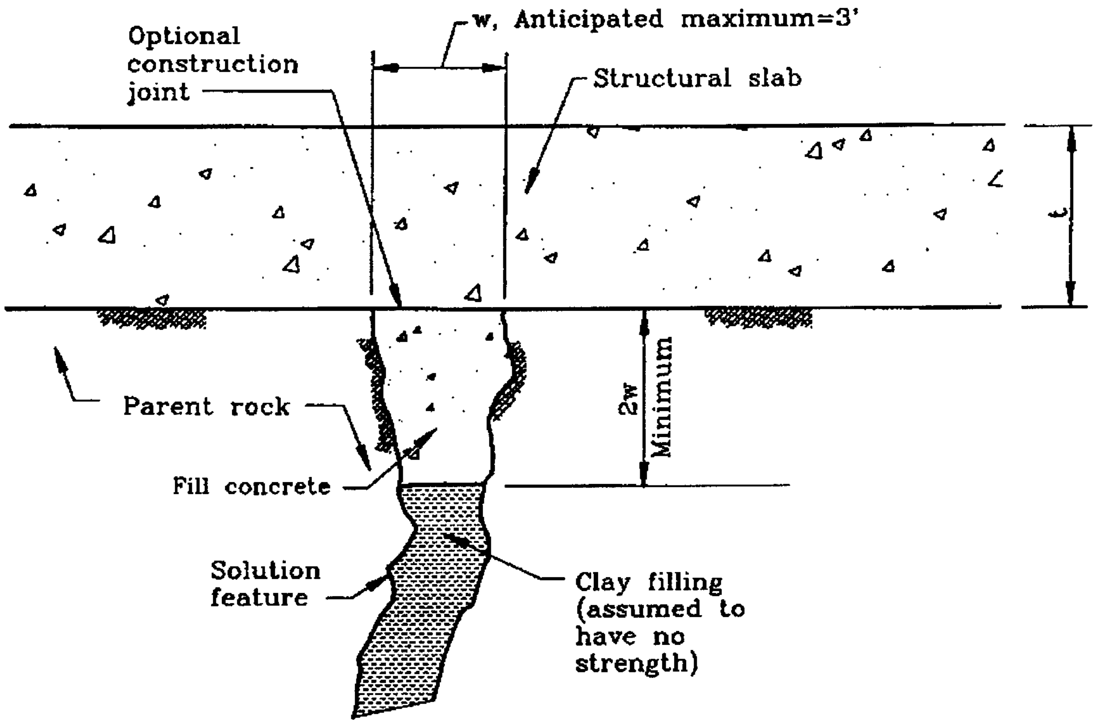
The case of swelling or squeezing rock represents yet another special problem. In such cases the rock foundation changes after it is exposed or unloaded, and the rock expands (increases in apparent volume) horizontally or vertically. There are at least five mechanisms which can cause swelling rock. Swelling may be result of a single mechanism or a combination of several interacting mechanisms. The five common mechanisms of swelling rock include elasto-plastic rebound (or heave), cation hydration, chemical reaction, loss of internal strength (creep), and frost action. Some of these mechanisms occur most commonly in certain rock types. Each category is discussed individually.

12-6. Rebound

Elasto-plastic rebound is the expansion of rock due to the reduction or removal of external forces acting upon the rock mass. In some cases, especially in areas with a high horizontal stress field, removal of as little as a few feet of rock or soil may result in an expansion of the exposed rock. The expansion may be expressed as a general heave of the exposed rock or as a pop-up or buckling. This behavior frequently occurs in areas associated with glacial activity and can occur in most types of rock. In structural excavations where rebound may be a problem, the surface may be rapidly loaded with a weight equivalent to the overburden to prevent rebound of the rock. In many nonstructural open cut excavations, this type of swelling may be more of a minor maintenance problem than a serious concern.

12-7. Cation Hydration

Cation hydration is another mechanism for swelling that is most frequently associated with some argillaceous rocks.



NOTES:
1. CLEAN ALL SOLUTION FEATURES TO A MINIMUM DEPTH OF 2w. IF FEATURE WIDENS WITH DEPTH, CLEAN TO A DEPTH WHERE WEDGING CAN BE ACHIEVED. FINAL DEPTH OF CLEANING FOR ALL FEATURES TO BE COORDINATED WITH PROJECT GEOLOGIST.
2. DEGREE OF CLEANING TO BE THE SAME AS FOR HORIZONTAL FOUNDATION SURFACES.
3. IN THE EVENT THAT A ZONE OF NUMEROUS CLOSELY SPACED SOLUTION FEATURES(NO INDICATION OF THIS FROM DRILLING), w SHOULD BE TAKEN AS THE ZONE WIDTH.
4. FEATURES LESS THAN 3" WIDE CAN BE DISREGARDED EXCEPT FOR NORMAL FOUNDATION CLEANUP.

Figure 12-3. Criteria for treatment of solution-widened joints

The process refers to the attraction and adsorption of water molecules by clay minerals. Factors that contribute to this form of swell include poor cementation, desiccation and rewetting, unloading, and high clay mineral content; especially montmorillonite clay.

a. Problem rocks. Clay shale is the rock most commonly associated with swelling problems, and its principal mechanism of swell is cation hydration. Defined as shales that tend to slake easily with alternate wetting and drying, clay shales were overconsolidated by high loads in the past. Typically, clay shales were deposited in shallow marine or deltaic environments in Cretaceous or Paleocene times and contain a high percentage of swell prone montmorillonite mineral.

b. Other factors. Factors other than clay mineral type and content may also contribute to cation hydration induced swell. These factors include density, moisture content, rock mineral structure, loading history, and weathering.

(1) Density of the rock is an important indicator of swell. A 25 percent increase in the dry density of clay shales can more than double the maximum swell pressures developed in the material. Therefore a high density could indicate high swell pressure potential.

(2) Low moisture content can indicate a high swell potential, since there is more availability for water within the clay structure.

(3) The mineral structure of the clay shale can influence the magnitude and isotropy of the swell characteristics. A compacted mineral orientation typical of clay shales has most of the plate mineral faces in a "stacked" arrangement, with maximum swell potential normal to the mineral faces.

(4) The loading history can indicate the degree of preconsolidation that the shales have been subjected to in the past. Changes in the stress environment can be due to erosion, glaciation, stream downcutting, and engineering activities.

(5) Weathering of clay shales generally reduces the swell potential unless additional expansive clays are formed.

c. Excavation problems. Excavations in clay shale present special problems. If the excavated surface is allowed to dry, the material develops shrinkage cracks, and rebound-type swell induces a relative moisture reduction and density decrease. Water or moisture from concrete applied to this surface can induce swelling of the clay shale. Slope stability is another prominent problem in clay shales, since any excavation can result in renewed movement along older, previously stable slide planes. The presence of unfavorably oriented bentonite seams common in clay shales can present serious stability hazards.

d. Treatment methods. Preventive measures can include careful control of the excavation sequence, moisture control and surface protection, and favorable stratigraphic placement and orientation of the slopes and structures. Treatment methods are discussed in Chapter 11.

e. Field investigations. Field investigations should include checks for significant problem prone clay shale formations, some of which are listed in Table 12-3. Also, some indications of clay shale swell problems include hummocky terrain along river valley slopes, slides along road cuts, and tilting or cracking of concrete slabs or light structures. Slickensides in shale is another indicator of swelling potential. The presence of clay shales susceptible to cation hydration swelling in the project region should be determined very early in the exploration program.

f. Laboratory tests. Laboratory tests to determine engineering properties are similar to those for soil mechanics, and include clay mineral type and percentage analyses, Atterberg limits, moisture content, consolidation tests, and swell tests. In decreasing order, the significant swell producing clay minerals are montmorillonite, illite, attapulgite, and kaolinite. Atterberg limit tests can indicate the swelling nature of clay shale, with high plasticity indices correlating to high swell potential, as shown in Table 12-4. The method of determining the Atterberg limits of clay shale must be consistent, since air drying, blending, or slaking of the original samples may provide variable results. There are several methods of performing consolidation and swell tests on clay shales. These methods are summarized in Table 12-5.

12-8. Chemical-Reaction Swelling

Chemical-reaction swelling refers to a mechanism most commonly associated with Paleozoic black shales such as the Conemaugh Formation, or the Monongahela Formation in Pennsylvania. Swell develops when reactions such as hydration, oxidation, or carbonation of certain constituent minerals create by-products that results in volumes significantly larger than the original minerals. These reactions can result in large swelling deformations and pressures after excavation and construction. The conditions that are conducive to this type of swelling may not occur until after a foundation is in place, and similar conditions may not be reproduced easily in the laboratory to indicate that it may be a problem. Temperature, pressure, moisture, adequate reactants and, in some cases, bacterial action are critical parameters for reaction to occur.

a. Reactions. The transformation of anhydrite to gypsum is one of the more common reactions. In shales containing a substantial percentage of free pyrite, a similar reaction can occur. The oxidation of the pyrite can result in the growth of gypsum crystals or a related mineral, jarosite. The presence of sulphur bacteria can aid the reaction, and may be essential to the reaction in some cases. Sulfuric acid produced by the reaction may react with any calcite in the shale to increase the development of gypsum. The resulting growth of gypsum crystals causes the swelling, which can uplift concrete structures.

b. Treatment methods. Since the reactions are difficult to predict or simulate during exploration and design, it may be desirable to avoid placing structures on pyrite-bearing carbonaceous shales of Paleozoic age, since these rocks are the most common hosts for chemical-reaction swelling. If avoidance is not an option, the exposed surfaces may be protected from moisture changes by placing a sealing membrane of asphalt or some other suitable material. Shotcrete is not a suitable coating material since the sulfuric acid produced by the reaction can destroy it. The added source of calcium may even enhance the swelling reaction. Another preventive measure may be the application of a chemical additive which blocks the growth of gypsum crystals. Tests have indicated that

Table 12-3
Landslide-Susceptible Clay Shales in United States

Stratigraphic Unit	Description
Bearpaw shale	Upper Cretaceous; northern, eastern and southern Montana, central northern Wyoming, and southern Alberta, Canada; marine clay shale 600 to 700 ft; in Montana group.
Carlile shale	Upper Cretaceous; eastern Colorado and Wyoming, Nebraska, Kansas, and South Dakota, southeastern Montana and northeastern New Mexico; shale, 175 to 200 ft; in Colorado group.
Cherokee shale	Early Pennsylvanian; eastern Kansas, southeastern Nebraska, northwestern Missouri and northeastern Oklahoma; shale, 500 ft; in Des Moines group.
Claggett formation	Upper Cretaceous; central and eastern Montana, and central northern Wyoming; marine clay shales and sandstone beds, 400 ft; in Montana group.
Dawson formation	Upper Cretaceous-Lower Tertiary; central Colorado; nonmarine clay shales, siltstone and sandstone, 1000 ft.
Del Rio clay	Lower Cretaceous; southern Texas; laminated clay with beds of limestone; in Washita group.
Eden group	Upper Ordovician; southwestern Ohio, southern Indiana, and central northern Kentucky; shale with limestone, 250 ft; in Cincinnati group.
Fort Union group	Paleocene; Montana, Wyoming, North Dakota, northwestern South Dakota, and northwestern Colorado; massive sandstone and shale, 4000 ft +.
Frontier formation	Upper Cretaceous; western Wyoming and southern Montana; sandstone with beds of clay and shale, 2000 to 2600 ft; in Colorado group.
Fruitland formation	Upper Cretaceous; southwestern Colorado and northwestern New Mexico; brackish and freshwater shales and sandstones, 194 to 530 ft; late Montana age.
Graneros shale	Upper Cretaceous; eastern Colorado and Wyoming, southeastern Montana, South Dakota, Nebraska, Kansas and northeastern New Mexico; argillaceous or clayey shale, 200 to 210 ft; in Colorado group.
Gros Ventre formation	Middle Cambrian; northwestern Wyoming and central southern Montana; calcareous shale with conglomeratic and oolitic limestone, 800 ft.
Jackson group	Upper Eocene; Gulf Coastal Plain (southwestern Alabama to southern Texas); calcareous clay with sand, limestone, and marl beds.
Mancos shale	Upper Cretaceous; western Colorado, northwestern New Mexico, eastern Utah, southern and central Wyoming; marine, carbonaceous clay shale with sand, 1200 to 2000 ft; of Montana and Colorado age.
Merchantville clay	Upper Cretaceous; New Jersey; marly clay, 35 to 60 ft; in Matawan group.
Modelo formation	Upper Miocene; southern California; clay, diatomaceous shale, sandstone, and cherty beds, 9000 ft.
Monterey shale	Upper, middle and late lower Miocene; western California; hard silica-cemented shale and soft shale, 1000 ft +.
Morrison formation	Upper Jurassic; Colorado and Wyoming, south central Montana, western South Dakota, western Kansas, western Oklahoma, northern New Mexico, northeastern Arizona, and eastern Utah; marl with sandstone and limestone beds, 200 ft +.
Mowry shale	Upper Cretaceous; Wyoming, Montana and western South Dakota; hard shale, 150 ft; in Colorado group.
Pepper formation	Upper Cretaceous; eastern Texas; clay shale.
Pierre shale	Upper Cretaceous; North Dakota, South Dakota, Nebraska, western Minnesota, eastern Montana, eastern Wyoming, and eastern Colorado; marine clay shale and sandy shale, 700 ft; in Montana group.
Rincon shale	Middle or lower Miocene; southern California; clay shale with lime stone, 300 to 2000 ft.
Sundance formation	Upper Jurassic; southwestern South Dakota, Wyoming, central southern Montana; northwestern Nebraska; and central northern Colorado; shale with sandstone, 60 to 400 ft.
Taylor marl	Upper Cretaceous; central and eastern Texas; chalky clay, 1200 ft.
Thermopolis shale	Upper Cretaceous; central northern Wyoming, and central southern Montana; shale with persistent sandy bed near middle, 400 to 800 ft; in Colorado group.
Trinity group	Lower Cretaceous; Texas, south central and southeastern Oklahoma, southwestern Arkansas, and northwestern Louisiana; fine sand, gypsiferous marl and occasional limestone.
Wasatch formation	Lower Eocene; Wyoming, south central and eastern Montana, southwestern North Dakota, western Colorado, Utah, and northwestern New Mexico; sands and clay, 0 to 5000 ft +.

Table 12-4
Swell Potential and Atterberg Limits

Index Property	Swelling Potential Low	Medium	High
Liquid Limit	30-40	40-55	55-90
Plastic Limit	15-20	20-30	30-60
Shrinkage Limit[1]	35-25	25-14	14-8
Free Swell[2]	20-40	40-70	70-180

Notes:
1. Poor Correlation to Swelling Properties.
2. Described by Katzir and David (1986).

diethylenetriamine penta (methylene phosphonic acid), substantially inhibited gypsum development under normally reactive conditions. Other similar crystal growth inhibitors may be useful in preventing chemical-reaction swelling.

12-9. Loss of Internal Strength

This swelling mechanism occurs most commonly when intact rock loses its internal bonding or cementation. The mechanism is commonly associated with extensive alteration in major faults occurring in granites, gneisses, and poorly-cemented sandstones under stress conditions commonly associated with tunneling projects, but it may be of concern in very deep, open excavations. The swelling acts primarily on side-walls as a type of slow continuous plastic deformation under a constant load. Problems caused by this swell mechanism are usually of more concern where close tolerances and long-term stability are critical.

12-10. Frost Action

Freezing can induce swelling or heaving of rock in excavations by the expansion of water within the rock mass. Although pore water freezing in porous rocks may be of some concern, the principal concern is freezing water in joints, bedding planes, and other openings in the rock. Since many of these discontinuities may have been relatively tight prior to freezing, a spalling effect from frost may induce a nonrecoverable bulking of the rock and reduction in strength of the rock mass in addition to the temporary uplift by freezing. Preventive measures can include limiting excavation of final grades to warmer seasons, moisture controls or barriers, and layers of soil or insulation blankets in areas of special concern.

12-11. Design Considerations

If rock in an excavation is found to have a swelling potential, it may not be a serious concern unless structures are to be placed on the rock surface. With structures, swell and differential swell must then be considered and preventive techniques used. Some foundation design techniques for handling swell problems are summarized in Table 12-6.

Section III
Soil-Rock Contacts

12-12. General

Some of the most difficult excavation problems occur in rock that has been severely weathered or altered. While it is generally assumed that bedrock will be easy to locate and identify, the assumption may not always be correct. In some cases, weathering can form a residual soil that grades into unweathered bedrock, with several rock-like soil or soil-like rock transitions in between. These residual soils, saprolites, and weathered rocks require special consideration, since they may have characteristics of both rock and soil which affect rock excavations and foundation performance.

12-13. Weathering Profiles

Chemical weathering is the primary cause of gradational soil-rock contacts, with the most prominent cases occurring in warm, humid climates. The result can be irregular or pinnacled rock covered by gradational materials composed of seamy, blocky rock, saprolite, and soil. The preferred case of an abrupt contact between soil and unweathered rock is not usually what is found. General descriptions of the zones in typical profiles for igneous and metamorphic rocks are given in Table 12-7. There are similarities in the development of these profiles. Weathering tends to dissolve the most soluble materials and alter the least stable minerals first, following rock mass discontinuities such as faults, joints, bedding planes, and foliations. The unweathered rock surface may be highly irregular due to solution and alteration along these openings. Engineering design and excavation considerations are dependent upon specific weathering profiles developed in certain rock types. These profiles include: massive igneous, extrusive igneous, metamorphic, carbonate, and shale.

a. Massive intrusive igneous profiles. Rocks typical of massive igneous profiles include granites and other

Table 12-5
Summary of Swell Potential Tests

Test Method	Test Procedure Summary	Remarks
Free-Swell Test	This specimen is over-dried, granulated and placed in a test tube. Water is added and the amount of volume expansion is recorded.	The rock structure is destroyed and the grain sizes are reduced.
Calculated-Pressure Test	An intact specimen is immersed in kerosene or mercury to determine its initial volume. The specimen is then placed in water and allowed to swell. If the specimen remains intact, the new volume can be determined by again immersing the specimen in mercury. Otherwise, the swell is recorded as the change in volume of the water-specimen system.	The loading and confining pressures are not representative of in-situ conditions.
Unconfined Swell Test	The specimen is placed in a container and ames dials are set to one or more axes of the specimen. Water is added and the axial expansion is recorded.	The loading and confining pressures are not representative of in-situ conditions.
Nominal Load Test	A specimen is inserted in a consolidometer, a nominal seating load is applied (generally 200 psf or 0.10 Kg/cm^2) and water added. The volume change is recorded by an ames dial.	The loading and confining pressures are not representative of in-situ conditions.
Calculated-Pressure Test	The specimen placed in a consolidometer and subjected to a calculated overburden pressure. Free access to water is then permitted and the volume expansion recorded. Modifications of this test included rebounding the specimen to the original void ratio.	
Constant-Volume Test	The specimen is inserted in a consolidometer and a seating load applied. Water is added, and pressure on the specimen increased such and pressure on the specimen increased such that the total volume change of the specimen is zero. The final pressure is taken as the "swell pressure".	Under in-situ conditions, the volume may change resulting in a reduced final pressure.
Double-Deadmeter Test	Two similar specimens are place in separate consolidometers. One specimen is subjected to calculated overburden pressures and the deformation recorded. The other specimen is allowed free access to water, is permitted to swell and then is subjected to overburden pressure. The difference in deformations or strain of the two specimens at the overburden pressure is considered the potential swell of the material.	Results are more typical of field conditions.
Triaxial Test	A specimen is consolidated to the in-situ pressure as evaluated by stress measurements of statistical analyses. Generally, the horizontal stress is greater than the applied vertical stress. The specimen is then allowed free access to water and the swell recorded.	This test is typical of field conditions, and can simulate high horizontal stress field.

Table 12-6
Design Techniques and Methods of Treatment for Swelling Rocks (after Linder 1976)

a. Waterproofing Below and Around Foundations: Though successful in preventing drainage into the strata directly below the foundation, this method does not consider evaporation. The technique is best employed to prevent desiccation of strata during construction.

b. Rigid-Box Design: The design of the foundation into separate reinforced concrete units or boxes that can withstand predicted stresses and deformations is a feasible yet expensive solution to swell.

c. Saturation and Control: Saturation of swell-susceptible strata before construction by ponding will help reduce swell after construction. However, if the water content is not maintained additional settlement will be experienced during the life of the structure. Also seasonal fluctuation of the availability of water may cause the structure to rise and fall periodically.

d. High Loading Points: As swell is a function of both deformation and pressure, it was reasoned that foundations with high unit loadings should experience less swell. However, such foundations have met numerous problems including uplift on footings. Such foundations also influence only a small volume below the footing and swell may be experienced due to swell of deeper strata.

e. Replacement of the Stratum: A drastic, expensive, yet totally effective procedure for near-surface strata.

f. Piers: The concept of placing the base of the foundation below swell susceptible strata or where water content changes are expected to be minimal has also been employed with varying success. Problems such as side friction and water changes induced by construction must be considered.

g. Flexible Construction: For light structures, the division of the structure into units which can move independently of each other can be a practical solution. Differential heave between units will cause no stress to the structure and minor repair work will assure continuing service.

h. Raised Construction: A little-used alternative is to place the structure on a pile system raised above the surface. This would allow normal air circulation and evaporation below the structure and if drainage is properly designed should cause minimal disturbance to the water content of swell susceptible strata.

igneous rocks with relatively homogenous, isotropic texture. Since this type of rock has few or no bedding planes, foliations, or concentrations of minerals relatively susceptible to weathering, the existing joints, faults, and shear zones control the development of weathering. Stress-relief slabbing or sheeting joints subparallel to the ground surface also provide a path for chemical weathering, as shown in Figure 12-4. Saprolite (Zone IC in Figure 12-4) in this type of profile may retain the texture and orientation of the parent rock. Relict joints may still act as sliding-failure planes or preferred paths for ground water flow, so some of the parent rock's properties still apply to this material. The transition (Zone IIA) has the same slide failure and ground water concerns as with the saprolite, but the element of corestones becomes an additional concern. These are the hard, partially weathered spheroidal centers of blocks that can range from soft to relatively hard, and from small size to relatively large. The transition zone may require a modification of the excavation methods used, from purely mechanical soil excavation methods to the occasional use of explosives or hand-breaking. Corestones in this type of profile are generally spheroidal, which can cause difficulties in excavation and removal if they are relatively large.

b. Extrusive igneous profiles. Extrusive rocks such as basalt develop profiles and conditions similar to those found in massive igneous rocks. However, certain structural features common in basalts and tuffs make conditions extremely variable in some areas. For example, lava flow tubes and vesicular basalt may increase the weathering path in some zones. The nature of flow deposits may make rock conditions in excavations difficult to predict since there may be buried soil profiles and interbedded ash falls or tuffs which are more permeable than adjacent basalts. These complex permeable zones can increase weathering and store water under relatively high pressures. Also, soils in the upper horizons may have unpredictable engineering characteristics due to unusual clay minerals present from the weathering of highly ferromagnesian parent materials.

c. Metamorphic profiles. Since the structure or texture of metamorphic rocks can range from schistose to nearly massive gneissic, the weathering profiles can vary greatly, as illustrated in Figure 12-5. Foliations in the rocks and changes of the lithology enhance the variability that can be found in the weathering profiles in metamorphics. The results are differences in the depths

Table 12-7
Description of a Typical Weathering Profile

	Zone	Description	RQD[1] (NX Core, percent)	Percent Core Recovery[2] (NX Core)	Relative Permeability	Relative Strength
I Residual Soil	1A-A Horizon	- top soil, roots, organic material zone of leaching and eluviation may be porous	--	0	medium to high	low to medium
	1B-B Horizon	- characteristically clay-enriched also accumulations of Fe, A1 and Si hence may be cemented - no relict structures present	--	0	low	commonly low (high if cemented)
	1C-C Horizon	- relict rock structures retained - silty grading to sandy material - less than 10 percent core stones - often micaceous	0 or not applicable	generally 0-10 percent	medium	low to medium (relict structures very significant
II Weathered Rock	IIA-Transition (from residual soil or saprolite to partly weathered rock)	- highly variable, soil-like to rock-like - fines commonly fine to coarse sand (USS) - 10 to 90 percent core stones - spheroidal weathering common	variable, generally 0-50	variable, generally 10-90%	high (water losses common)	medium to low where waste structures and relict structures are present
	IIB-Partly weathered rock	- rock-like, soft to hard rock - joints stained to altered - some alteration of feldspars and micas	generally 50-75 percent	generally >90 percent	medium to high	medium to high[2]
III Unweathered Rock		- no iron stains to trace long joints - no weathering of feld- and micas	>75 percent (generally >90 percent)	generally 100 percent	low to medium	very high[2]

Notes:
1. The descriptions provide the only reliable means of distinguishing the zones.
2. Considering only intact rock masses with no adversely oriented geologic structure.

of weathering profiles developed over each lithology, in some cases up to 50 meters of difference vertically in just a few feet horizontally (Deere and Patton 1971). Intrusive dikes commonly found in metamorphic terrains may either be more or less resistant to weathering than the surrounding rock, forming either ridges or very deep weathering profiles. Problems in this type of profile include slide instability along relict foliation planes, highly variable depth to unweathered bedrock, and potentially high-pressure ground water storage in faults or behind intrusive dikes.

d. Carbonate profiles. Carbonate rock weathering was previously discussed in relation to karst development within the rock mass. The same weathering conditions may affect the surface of the rock. Carbonate rocks

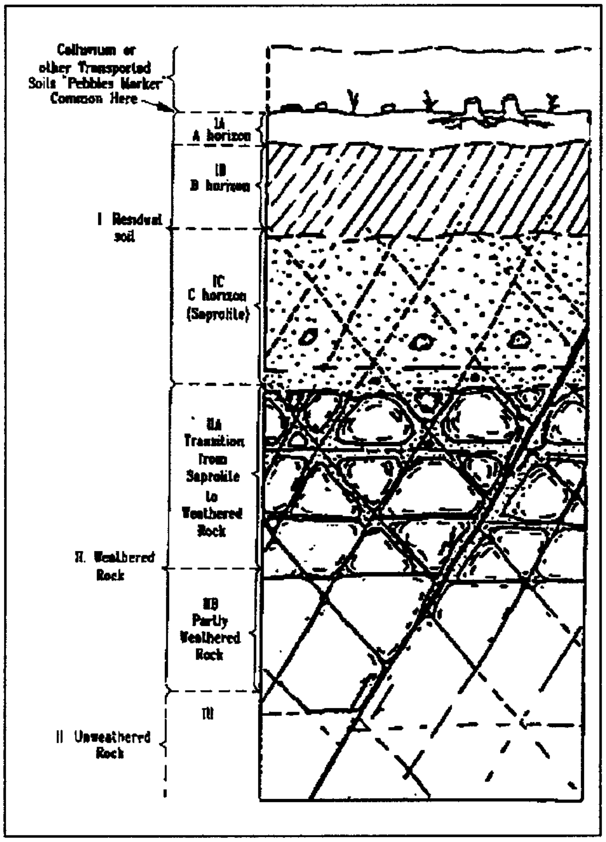


Figure 12-4. Typical weathering profile for intrusive igneous rocks (from Deere and Patton 1971)

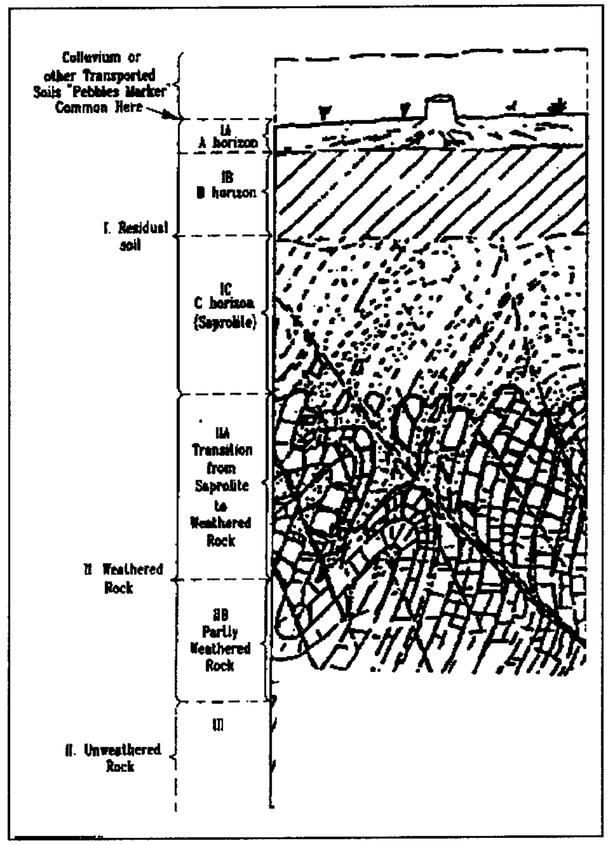


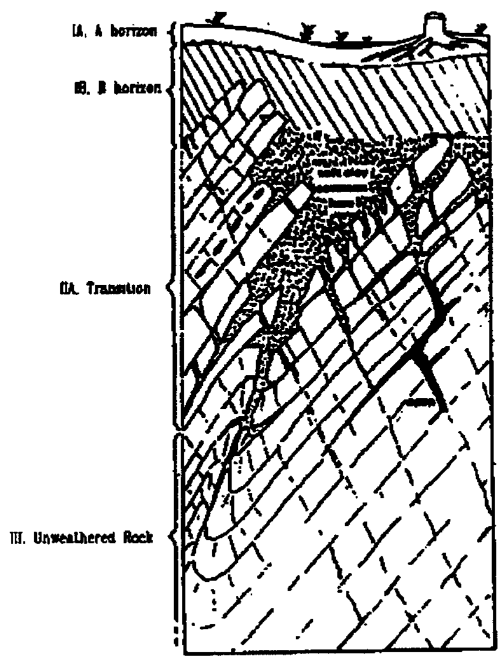
Figure 12-5. Typical weathering profile for metamorphic rocks (from Deere and Patton 1971)

develop into a profile, as illustrated in Figure 12-6, with sharp contacts between soils and weathered rock, unlike igneous, and metamorphic profiles. Occasionally, carbonate rocks may have chert, sand, or clay which form saprolite and retain a relict structure upon weathering. In most cases, however, the carbonates are removed and the remaining insoluble residue, typically a dark red clayey "terra rosa," lies directly upon weathered rock. A jagged, pinnacled rock surface may develop due to weathering along faults or near-vertical joints. Troughs between the peaks may contain soft, saturated clays called "pockets of decalcification." Construction problems may include clayey seams, soft clays, rough bedrock surface, unstable collapse residuum, and rock cavities.

e. Shale profile. Shale weathering profiles also develop primarily along joints and fissures, but the weathering profile is generally thinner and the transition from soil to unweathered rock tends to be more gradual. Shale is generally composed of minerals which are the weathering by-products of other rocks, so under a new weathering environment they are not affected to the extent other rocks are. Mechanical weathering mechanisms, such as drying and rewetting, freeze-thaw cycles, and stress relief play a more important role in the development of a shale weathering profile, so increased fracturing is the characteristic of increasingly weathered shale. Interbedded sandstones tend to make the weathering patterns and overall stability problems more complex. For engineering design purposes, the handling of shale excavations grades from rock mechanics into soil mechanics, where most weathered shales can be treated as consolidated clays.

12-14. Design Considerations in Weathering Profiles

During subsurface investigations, saprolites most likely are classified as soils, since the samples recovered by subsurface drilling programs frequently end up as a



NOTES:
1) Very impure (sandy or silty) carbonates may develope a saprolite, IC zone.
2) A partly weathered chalky limestone, IIB zone is sometimes present.

Figure 12-6. Typical weathering profile for carbonate rocks (from Deere and Patton 1971)

disaggregated, crumbly material with no apparent structure. The sampling technique frequently destroys the interparticle bonding and gives the designer a poor idea of the actual conditions. Care should be taken during sampling to determine if saprolites and relict structures exist if they will be exposed in rock excavations. Trenching provides a better picture of the weathering profile in critical areas.

a. Saprolites. Since relict discontinuities may exist in saprolite zones, sliding or toppling of weak blocks may be difficult to evaluate in stability analyses. In some cases, studies using key-block theory (Goodman and Shi 1985) may be applicable to saprolites. The discontinuities may also be the principal permeability path for ground water in saprolites, and water pressure in relict joints may play a substantial part in excavation stability. For design purposes, saprolites should be considered a weak, blocky, seamy rock in which discontinuities govern the behavior. For excavation purposes, saprolites may be treated as a firm soil, requiring standard soil excavation techniques.

b. Transition materials. Below the saprolites in the weathering profile, the nature of the materials is more difficult to determine. The materials may act as a soil matrix with rock fragments of lesser importance, a rock mass with soil-like, compressible seams, or some intermediate material. The primary concern is the thickness compressibility or stability of the soil-like material between core-blocks or in seams, which governs the behavior of the material to a larger degree than the more easily recovered competent rock. In addition, rock in these zones may have an irregular surface, but may be adequate for load bearing. These conditions may require removal of all pinnacles to a prescribed suitable depth, or cleaning out of the crevices and backfilling with dental concrete. Lightly loaded footings on seamy rock may be adequate if the footings are expanded to prevent eccentric loading on individual blocks. If settlements are anticipated to be excessive using these techniques, drilled piers extending to competent rock at depth may be an economic alternative.

Appendix A
References

A-1. Required Publications

TM 5-232
Elements of Surveying, Headquarters, Department of the Army

TM 5-235
Special Surveys, Headquarters, Department of the Army

TM 5-818-1/AFM 88-3 (Chapter 7)
Procedures for Foundation Design of Buildings and Other Structures

ER 1110-1-1801
Construction Foundation Report

ER 1110-1-1802
Provision for Spacers to Show Voids and Core Losses in Core Samples and Requirements for Photographic Record of Cores

ER 1110-2-112
Required Visits to Construction Sites by Design Personnel

ER 1110-2-1806
Earthquake Design and Analysis for Corps of Engineers Projects

EP 1110-1-10
Borehole Viewing Systems

EM 385-1-1
Safety and Health Requirements Manual

EM 1110-1-1802
Geophysical Exploration

EM 1110-1-1804
Geotechnical Investigations

EM 1110-1-1904
Settlement Analysis

EM 1110-1-2907
Rock Reinforcement

EM 1110-2-1902
Stability of Earth and Rockfill Dams

EM 1110-2-1907
Soil Sampling

EM 1110-2-1908 (Part 1 of 2)
Instrumentation of Earth and Rockfill Dams (Ground Water and Pore Pressure Observations)

EM 1110-2-1908 (Part 2 of 2)
Instrumentation of Earth and Rockfill Dams (Earth Movement and Pressure Measuring Devices)

EM 1110-2-2200
Gravity Dam Design

EM 1110-2-2502
Retaining and Flood Walls

EM 1110-2-3504
Chemical Grouting

EM 1110-2-3506
Grouting Technology

EM 1110-2-3800
Systematic Drilling and Blasting for Surface Excavations

EM 1110-2-4300
Instrumentation for Concrete Structures

A-2. Related Publications

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Appendix B
Rock Mass Classification Tables

Table B-1
Input Data Sheet for the Geomechanics (RMR) Classification System

CLASSIFICATION INPUT DATA WORKSHEET: GEOMECHANICS CLASSIFICATION OF ROCK MASSES

Name of project: ____________
Site of survey: ____________
Conducted by: ____________
Date: ____________

STRUCTURAL REGION

No. ____________
Sta. ____________
Sta. ____________
Sta. ____________
Sta. ____________

ROCK TYPE AND ORIGIN

DRILL CORE QUALITY R.Q.D.

Excellent:	90-100%	....
Good:	75- 90%	....
Fair:	50- 75%	....
Poor:	25- 50%	....
Very poor:	< 25%	....
Range %:		....

WALL ROCK OF DISCONTINUITIES

Unweathered ....
Slightly weathered ....
Moderately weathered ....
Highly weathered ....
Completely weathered ....

GROUNDWATER

INFLOW per 1,000 ft of tunnel length gal/min ....
or
GENERAL CONDITIONS (completely dry, damp, wet, dripping or flowing under low, medium or high pressure):
....

STRENGTH OF INTACT ROCK MATERIAL

	Uniaxial compressive strength, psi	
Very high:	Over 32,000	....
High:	16,000 - 32,000	....
Medium:	8,000 - 16,000	....
Low:	4,000 - 8,000	....
Very low:	150 - 4,000	....

CONDITION OF DISCONTINUITIES

		Set 1	Set 2	Set 3
CONTINUITY				
Very low:	< 3 ft	....	....	....
Low:	3-10 ft	....	....	....
Medium:	10-30 ft	....	....	....
High:	> 30 ft	....	....	....
SEPARATION				
Tight joints:	< 0.01	....	....	....
Moderately open joints:	0.01-0.1 in.	....	....	....
Open joints:	0.1-0.5 in.	....	....	....
Range in.:		....	....	....
ROUGHNESS				
Very rough surfaces:		....	....	....
Rough surfaces:		....	....	....
Slightly rough surfaces:		....	....	....
Smooth surfaces:		....	....	....
Slickensided surfaces:		....	....	....
FILLING (GOUGE)				
Type:		....	....	....
Thickness:		....	....	....
Consistency:		....	....	....

SPACING OF DISCONTINUITIES

		Set 1	Set 2	Set 3
Very wide:	Over 10 ft	....	....	....
Wide:	3-10 ft	....	....	....
Moderate:	1-3 ft	....	....	....
Close:	2 in. - 1 ft	....	....	....
Very close:	< 2 in.	....	....	....
Range:		....	....	....

MAJOR FAULTS OR FOLDS

Describe major faults and folds specifying their locality, nature, and orientations.

STRIKE AND DIP ORIENTATIONS

				DIRECTION
Set 1	Strike:	(from .... to ....)	Dip:	....
Set 2	Strike:	(from .... to ....)	Dip:	....
Set 3	Strike:	(from .... to ....)	Dip:	....

GENERAL REMARKS AND ADDITIONAL DATA

The geologist should supply any further information which he considers relevant.

Table B-2
Geomechanics Classification of Jointed Rock Masses

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

<table>
<tr><th colspan="3">PARAMETER</th><th colspan="7">RANGES OF VALUES</th></tr>
<tr><td rowspan="3">1</td><td rowspan="2">Strength of intact rock material</td><td>Point-load strength index</td><td>> 10 MPa</td><td>4 - 10 MPa</td><td>2 - 4 MPa</td><td>1 - 2 MPa</td><td colspan="3">For this low range - uniaxial compressive test is preferred</td></tr>
<tr><td>Uniaxial compressive strength</td><td>>250 MPa</td><td>100 - 250 MPa</td><td>50 - 100 MPa</td><td>25 - 50 MPa</td><td>5-25 MPa</td><td>1-5 MPa</td><td>< 1 MPa</td></tr>
<tr><td colspan="2">Rating</td><td>15</td><td>12</td><td>7</td><td>4</td><td>2</td><td>1</td><td>0</td></tr>
<tr><td rowspan="2">2</td><td colspan="2">Drill core quality RQD</td><td>90% - 100%</td><td>75% - 90%</td><td>50% - 75%</td><td>25% - 50%</td><td colspan="3">< 25%</td></tr>
<tr><td colspan="2">Rating</td><td>20</td><td>17</td><td>13</td><td>8</td><td colspan="3">3</td></tr>
<tr><td rowspan="2">3</td><td colspan="2">Spacing of discontinuities</td><td>>2 m</td><td>0.6 - 2 m</td><td>200 - 600 mm</td><td>60 - 200 mm</td><td colspan="3">< 60 mm</td></tr>
<tr><td colspan="2">Rating</td><td>20</td><td>15</td><td>10</td><td>8</td><td colspan="3">5</td></tr>
<tr><td rowspan="2">4</td><td colspan="2">Condition of discontinuities</td><td>Very rough surfaces. Not continuous No separation Unweathered wall rock.</td><td>Slightly rough surfaces. Separation < 1 mm Slightly weathered walls</td><td>Slightly rough surfaces. Separation < 1 mm Highly weathered walls</td><td>Slickensided surfaces OR Gouge < 5 mm thick OR Separation 1-5 mm. Continuous</td><td colspan="3">Soft gouge > 5 mm thick OR Separation > 5 mm. Continuous</td></tr>
<tr><td colspan="2">Rating</td><td>30</td><td>25</td><td>20</td><td>10</td><td colspan="3">0</td></tr>
<tr><td rowspan="4">5</td><td rowspan="3">Ground water</td><td>Inflow per 10 m tunnel length</td><td>None</td><td><10 litres/min</td><td>10-25 litres/min</td><td>25 - 125 litres/min</td><td colspan="3">> 125</td></tr>
<tr><td>Ratio (joint water pressure / major principal stress)</td><td>OR 0</td><td>OR 0.0-0.1</td><td>OR 0.1-0.2</td><td>OR 0.2-0.5</td><td colspan="3">OR > 0.5</td></tr>
<tr><td>General conditions</td><td>OR Completely dry</td><td>OR Damp</td><td>OR Wet</td><td>OR Dripping</td><td colspan="3">OR Flowing</td></tr>
<tr><td colspan="2">Rating</td><td>15</td><td>10</td><td>7</td><td>4</td><td colspan="3">0</td></tr>
</table>

B. RATING ADJUSTMENT FOR JOINT ORIENTATIONS

Strike and dip orientations of joints		Very favourable	Favourable	Fair	Unfavourable	Very unfavourable
Ratings	Tunnels	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	-60

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

Rating	100 ← 81	80 ← 61	60 ← 41	40 ← 21	< 20
Class No	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

D. MEANING OF ROCK MASS CLASSES

Class No	I	II	III	IV	V
Average stand-up time	10 years for 15 m span	6 months for 8 m span	1 week for 5 m span	10 hours for 2.5 m span	30 minutes for 1 m span
Cohesion of the rock mass	> 400 kPa	300 - 400 kPa	200 - 300 kPa	100 - 200 kPa	< 100 kPa
Friction angle of the rock mass	> 45°	35° - 45°	25° - 35°	15° - 25°	< 15°

Table B-3
Summary of Joint Orientation Adjustments for Dam Foundations and Tunnels

A. Assessment of joint orientation favourability upon stability of dam foundations

Dip 0° - 10°	Dip 10° - 30°		Dip 30° - 60°	Dip 60° - 90°
	Dip direction			
	Upstream	Downstream		
Very favourable	Unfavourable	Fair	Favourable	Very unfavourable

B. The effect of joint strike and dip orientations in tunneling

Strike Perpendicular to Tunnel Axis			
Drive with Dip		Drive against Dip	
Dip 45°–90°	Dip 20°–45°	Dip 45°–90°	Dip 20°–45°
Very favorable	Favorable	Fair	Unfavorable

Strike Parallel to Tunnel Axis		Dip 0°–20° Irrespective of Strike
Dip 45°–90°	Dip 20°–45°	
Very unfavorable	Fair	Fair

Table B-4
Q-System: Description and Ratings for the RQD, J_n, J_r, J_a, and J_w Parameters (from Barton, Lien and Lunde 1974)

<u>Rock Quality Designation (RQD)</u>

Very poor................	0-25
Poor......................	25-50
Fair......................	50-75
Good......................	75-90
Excellent................	90-100

<u>Note</u>:

(i) Where RQD is reported or measured as ≤ 10 (including 0) a nominal value of 10 is used to evaluate Q in Eq. (1).

(ii) RQD intervals of 5, i.e. 100, 95, 90 etc. are sufficiently accurate.

<u>Joint Set Number (J_n)</u>

Massive, no or few joints	0.5-1.0
One joint set............	2
One joint set plus random	3
Two joint sets...........	4
Two joint sets plus random...................	6
Three joint sets.........	9
Three joint sets plus random...................	12
Four or more joint sets, random, heavily jointed, "sugar cube", etc........	15
Crushed rock, earthlike..	20

<u>Note</u>:

(i) For intersections use $(3.0 \times J_n)$

(ii) For portals use $(2.0 \times J_n)$

<u>Joint Roughness Number (J_r)</u>

(a) Rock wall contact and (b) Rock wall contact before 10 cms shear	
Discontinuous joints.....	4
Rough or irregular, undulating...............	3
Smooth, undulating.......	2
Slickensided, undulating	1.5
Rough or irregular, planar...................	1.5
Smooth, planar...........	1.0
Slickensided, planar.....	0.5
(c) No rock wall contact when sheared	
Zone containing cley minerals thick enough to prevent rock wall contact	1.0 (nominal)
Sandy, gravelly or crushed zone thick enough to prevent rock wall contact..................	1.0 (nominal)

<u>Note</u>:

(i) Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m.

<u>Note</u>:

(ii) J_r = 0.5 can be used for planar slickensided joints having lineation, provided the lineations are favorably orientated.

(iii) Descriptions B to G refer to small scale features and intermediate scale features, in that order.

(Continued)

Director, Defense Mapping Agency
ATTN: PR (MS A-13)
8613 Lee Highway
Fairfax, Virginia 22031-2137
Telephone: (703) 285-9339 or 9333

c. Recommended geospatial data standards. These are standards that are in various stages of development and which may become USACE standards in the future.

(1) Open Geodata Interoperability Standard/ Specification (OGIS).

The OGIS is a public domain software specification designed to promote true interchange among varied software and data structures. The OGIS is motivated by the costs of data conversion and sees independence from proprietary data structures. The OGIS project delivered a draft standards specification document in February 1994.

The proponent of the OGIS is the Open GIS Foundation (OGF) which may be reached at:

1 Kendall Square
Building 200, Suite 2200
Cambridge, MA 02139
Telephone: (617) 621-7025
FAX: (617) 621-7174

d. Guidelines.

(1) Guidelines for Implementing the National Geospatial Data Clearinghouse.

As defined in Executive Order 12906 the National Geospatial Data Clearinghouse is "a distributed network of geospatial data producers, managers and users linked electronically." Initially the clearinghouse functions are to publicize what geospatial data exists, the condition of these data and instructions on accessing the data. Later functions will provide direct access to the data, allow producers to publicize data that are prepared and planned and let users advertise their data needs. The clearinghouse will build on the Metadata Standard and the SDTS and will exploit the Wide Area Information Servers (WAIS).

The FGDC Clearinghouse Working Group developed the Clearinghouse and conducted a prototype test in the Fall of 1993. The EO 12906 requires the FGDC to establish a Clearinghouse within six months of the date of the order. The Clearinghouse is established and the Working Group is pursuing development of user guides, software enhancements and related support materials.

Chapter 8
USACE and Executive Order 12906

8-1. General

a. GD&S requirements at the Federal level revolve around the geospatial data itself. The unique character of geospatial data is recognized at the Federal level in Executive Order 12906, "Coordinating Geographic Data Acquisition and Access: The National Spatial Data Infrastructure." All of the actions Commands must take related to geospatial data have their regulatory basis in EO 12906. In this EO, several requirements related to the handling of geospatial data are instituted at the Federal level with the intent of reducing data duplication and protecting the nation's large investment in this class of information.

b. The EO places five (5) requirements on Federal agencies regarding the acquisition and access of geospatial data. These are: (1) document new geospatial data using the FGDC Content Standard for Digital Geospatial Metadata. (Data are considered new if produced or collected since January 1995.); (2) document existing geospatial data to the extent practicable. (Data are considered existing if collected or produced prior to January 1995.); (3) make documentation accessible to the Clearinghouse; (4) utilize the Clearinghouse prior to the collection or production of new geospatial data; (5) establish procedures to make geospatial data available to the public.

c. In ER 1110-1-8156, HQUSACE has developed a method for meeting these requirements. This method will promote consistency in the way USACE geospatial data holdings are documented and publicized and provide a practical method of meeting the requirements.

8-2. Document New Geospatial Data

a. EO 12906 SEC 3(b) requires agencies to thoroughly document the origins and characteristics of geospatial they collect or produce. The term "metadata" has been adopted to refer to this documentation. The first standard adopted by the FGDC describes the information that must be included in metadata.

b. Data documentation requirements depend on the age of the data. The EO states, "... Beginning 9 months from the date of this order [April, 1994], each agency shall document all new geospatial data ...." Thus data are considered new if produced or collected since January 1995. FGDC compliant metadata are required for all new data. Fully compliant metadata includes all of the mandatory and mandatory if applicable elements. As discussed in paragraph 7-3 of this EM, several considerations guide Commands in documenting new geospatial data.

c. Often, several simultaneous data collection efforts are launched at the beginning of a project in order to assemble background data or to populate a carefully designed database. The database may be composed of multiple files each containing a specific data theme or covering a specified geographic area of the same theme. Metadata for reach of these individual files will contain a lot of repetitive information. It may be appropriate to document groups of files that form a well defined dataset with a single metadata file.

d. The full text of "Content Standards for Digital Geospatial Metadata," (FGDC, 1994) is available via the Internet at ftp://waisqvarsa.er.usgs.gov/wais/docs. This standard is referred to here as the "metadata standard." Metadata normally reside in a text file distinct from the data they describe and can be consulted and shared easily. By examining metadata, potential users determine if existing geospatial data held by USACE meet their needs and decide if they wish to obtain a copy of the data. The characteristics of geospatial data that must be documented are established by the metadata standard. Using commercial and public domain software tools, FGDC compliant metadata can be developed in a straight forward manner.

8-3. Document Existing Geospatial Data

a. Eo 12906 calls for each agency to document older geospatial data to the extent practicable. The volume and diversity of existing geospatial data held by USACE Commands represents, without some consolidation, an insurmountable number of metadata files to be generated and subsequently managed. More importantly, much of the information about the origins, characteristics, and previous processing of these older data may be lost or difficult to find.

b. HQUSACE has developed a practical approach to consolidating and documenting older data. This approach employs the concept of geospatial data "collections" and the use of "minimum required metadata."

c. A data collection is a logically consistent grouping of geospatial data that can be documented in a uniform manner. Collections may be established based on a consistent theme or on a consistent spatial domain. Each collection can then be documented by a single "collection metadata" file containing only the mandatory elements of the metadata standard (primarily Sections 1 and 7 of the Metadata Standard). These minimum collection metadata are only to be used for existing (pre-1995) geospatial data.

d. For some collections of older data it may be possible to include other elements of the metadata standard as well (e.g., all the items in a collection have the same spatial data organization or the same spatial reference parameters). Collection metadata of older data rely heavily on a strongly descriptive abstract and a complete set of theme and place keywords to convey to the reader the origin and characteristics of the items included in the collection.

e. An example of a geospatial data collection based on theme is hydrographic survey data. A single Command may hold thousands of individual pre-1995 digital files containing hydrographic survey data. For documentation purposes, it is advantageous to consider the entire holding of older hydrographic survey data, regardless of age, format, or project location as a "data collection" and to document this collection with a single "collection metadata" file. Other examples of theme based collections are geodetic/survey control, geophysical logs, aerial photographs, and water control data. An example of a collection metadata file for hydrographic survey data is given in Table 8-1.

f. HQUSACE has developed collection metadata templates for data themes commonly held by Commands. These templates are available electronically via the USACE Node. Commands may define additional geospatial data collections as needed when documenting large holdings of existing (pre-1995) geospatial data. The usefulness of collection metadata is enhanced if Commands maintain an inventory of the individual items (files) represented by the collection if this can be accomplished with reasonable resources.

g. Data collections based on spatial domain generally pertain to a specific project or installation. These "project collections" may contain a variety of engineering, environmental and operations data in various formats generated during planning, construction and operation of the project. A project collection metadata file is given in Table 8-2.

h. Not all existing data can be included in a collection. Geospatial data produced prior to January 1995, if not included in a collection, can be documented as resources become available or as necessary for purposes of data exchange.

8-4. Make Documentation Accessible to the National Geospatial Data Clearinghouse

a. The third requirement of EO 12906 is for Federal Agencies to make geospatial data documentation electronically accessible. The National Geospatial Data Clearinghouse (Clearinghouse) was established for this purpose. The Clearinghouse is a distributed, electronically connected network of geospatial data producers, managers, and users. The Clearinghouse allows its users to determine what geospatial data exist, find the data they need, evaluate the usefulness of the data for their applications, and obtain or order the data as economically as possible. The Clearinghouse is functioning as an electronic geospatial data locator and access service operating on the Internet and is a key element of EO 12906.

b. As the overall agency response to this requirement, HQUSACE has established and maintains a centralized Clearinghouse Node (USACE Node) (electronic address) for all USACE Commands. The USACE Node functions as the primary point of public access to documentation about the USACE geospatial data holdings. A separate electronic data page for each USACE activity is maintained by HQUSACE on this server. The Internet URL is http://corps_geo1.usace.army.mil. This electronic site is for Clearinghouse and related purposes.

c. Commands have two actions related to making their metadata documentation accessible to the Clearinghouse. Commands must complete and submit collection metadata for older data as discussed in paragraph 8-3 above. The templates available on the server may be used for this purpose. Commands must submit the completed metadata for new geospatial data as discussed in paragraph 8-2 above.

d. Commands will utilize the USACE node for the purpose of presenting their geospatial data holdings to the public and will update their data pages not less than once a year. Commands will prepare and submit to the USACE node, all metadata as discussed in paragraphs 7.g(1), 7.g(2), and 7.g(3) of ER 1110-1-8156 and in paragraphs 8-2 and 8-3 of this manual. Commands will use the USACE Clearinghouse node for submission of all collection and full metadata. The node may also be utilized as a location for electronic data delivery if desired. Frequently requested geospatial data may be placed on the node for unrestricted direct public access as appropriate. The INTERNET URL address for the Corps Clearinghouse node is http://corps_geo1.usace.army.mil.

e. Commands may establish supplemental Clearinghouse nodes in accordance with CEIM guidance on requirements for computer network security. Commands must assure a link is maintained from the USACE node to any supplemental nodes established by the Command. Use of a supplemental node does not remove the requirement to provide and maintain all metadata on the USACE node.

f. The USACE node as established and maintained by HQUSACE serves several purposes: (1) The node estab-

lishes a single electronic site with specified formats and protocols for USACE Commands to list their data holdings; (2) The node provides a starting point and search engine for Commands to use when checking the Clearinghouse for existing data; (3) By completing the collection metadata examples for existing data Commands may fulfill the requirement to document existing geospatial data; (4) The node provides a location for public distribution of geospatial data for use by Commands if desired; (5) The node provides a focal point for distribution of information regarding GD&S issues in USACE.

8-5. Search the Clearinghouse

The fourth requirement of EO 12906 is for each agency to check the Clearinghouse for usable, available geospatial data before expending funds to collect new data. HQUSACE has chosen to enforce this requirement at the Command level. As set forth in ER 1110-1-8156, each Command must certify it has searched the Clearinghouse for available data as part of the budget submission process. The USACE node provides a form-tool to assist in searching the Clearinghouse.

8-6. Establish Procedures to Make Geospatial Data Available to the Public

Commands should establish internal procedures for responding to legitimate requests from the public for copies of geospatial data in the Command's holdings.

Table 8-1
Collection Metadata Describing New Orleans District Hydrographic Survey Data Collection

Identification_Information:

Citation:

Citation_Information:

Originator: U.S. Army Corps of Engineers, New Orleans District, New Orleans, Louisiana

Publication_Date: 19951031

Title: Hydrographic Survey Data Collection

Description:

Abstract: The U.S. Army Corps of Engineers (USACE) maintains a collection of hydrographic surveys that have been obtained in accordance with USACE Engineer Manual (EM) 1110-2-1003, "Hydrographic Surveying." USACE in-house crews and contractors collect this data using the traditional hydrographic surveying echo sounding equipment, as well as, airborne laser platforms. Hydrographic surveys are performed of navigation lock approach channels, in river and harbor navigation projects, and within the major inland waterway tributaries, such as the Mississippi River. Surveys may include revetment locations, project, river structures, and comprehensive river surveys. Some surveys are point-on-range surveys along range lines established at regular intervals in rivers and channels. The hydrographic survey collection is composed of digital data files that contain 3-dimensional coordinates defining river and harbor bottoms. These files contain real world coordinates and can be related to absolute positions on the earth. Sometimes the hydrographic surveys are merged with overbank topographic surveys to create a full model of the channel cross sections. Recently, hydrographic surveys have been conducted with sweep and multi-beam sonar survey equipment, rather than via conventional data collection. This technology will become increasingly common on future surveys. The New Orleans District began collecting hydrographic survey data in 1880 and began maintaining the data in digital form in 1980. The digital holdings represent over 1500 hydrographic survey jobs. The New Orleans District hydrographic survey data is collected along the major tributaries, waterways, coastal harbors, and engineering projects within the district such as the Mississippi River, Atchafaylaya River, the Gulf Intracoastal Waterway, and major streams, tributaries, and passes to the Gulf of Mexico and coastal Louisiana.

Purpose: Hydrographic surveys are conducted in support of planning, engineering, and design, construction, operation, maintenance, and regulation of civil works navigation and flood control projects. Specific example uses of the survey data include performing channel and levee slope stability analyses and construction of USACE structures, such as locks and dams, levees, and flood walls. Hydrographic surveys are also taken to monitor channel shoaling and scour to maintain channels at navigable project depths.

Time_Period_of_Content:

Time_Period_Information:

Range_of_Dates/Times:

  Beginning_Date: 1880

  Ending_Date: 1995

Currentness_Reference: ground condition

Status:

Progress: In work

Maintenance_and_Update_Frequency: Continually

Spatial_Domain:

Bounding_Coordinates:

West_Bounding_Coordinate:-94.25

East_Bounding_Coordinate:-88.67

North_Bounding_Coordinate:33.76

South_Bounding_Coordinate:28.28

Keywords:

Theme:

Theme_Keyword_Thesaurus: none

Theme_Keyword: hydrographic

Theme_Keyword_Thesaurus: none

Theme_Keyword: survey

Theme_Keyword_Thesaurus: none

Theme_Keyword: bathymetry

Place_Keyword_Thesaurus: Geographic Names Information System

Place_Keyword: Arkansas

Place_Keyword: Mississippi

Place_Keyword: Louisiana

Access_Constraints: None

Use_Constraints: These data were compiled for government use and represents the results of data collection/processing for a specific U.S. Army Corps of Engineers (USACE) activity. The USACE makes no representation as to the suitability or accuracy of these data for any other purpose and disclaims any liability for errors that the data may contain. As such, it is only valid for it is intended use, content, time, and accuracy specifications. While there are no explicit constraints on the use of the data, please exercise appropriate and professional judgment in the use and interpretation of these data.

(Continued)

Table 8-1 (Concluded)

Metadata_Reference_Information:
Metadata_Date: 19951031
Metadata_Contact: Chief, Surveys Section, Engineering Division
Contact_Information:
Contact_Organization_Primary:
Contact_Organization: U. S. Army Corps of Engineers, New Orleans District,

Contact_Address:
Address_Type: mailing address
Address:P.O. Box 60267
City: New Orleans
State_or_Province:LA>
Postal_Code: 70160-0267
Contact_Voice_Telephone: 504-865-1121

Metadata_Standard_Name: FGDC Content Standards for Digital Geospatial Metadata
Metadata_Standard_Version: 19940608

Table 8-2
Collection Metadata Describing the Kissimmee River Restoration Project Data Collection at the Jacksonville District

Identification_Information:
Citation:
Citation_Information:
Originator:

US Army Corps of Engineers
Jacksonville District
Jacksonville, Florida

Publication_Date: 19950901
Title: Kissimmee River Restoration Surveys

Description:
Abstract:
Digital orthophotography covering the Kissimmee River flood plain between Lake Kissimmee and Lake Okeechobee, was collected in support of the Kissimmee River Restoration (KRR). This collection consists of the orthoimagery, contours, spot elevations, cross sections, bathymetry and digital terrain models (DTMs). Some data is in ARC/INFO coverages and Grids, some in Intergraph design files, some in both. The flood plain of the Kissimmee is considered as divided into four pools along its length which correspond to structures along Canal 38. Each pool is further divided into blocks which serve to keep the file size of the data manageable. The grids, the contours, spot elevations, and other vector coverages are tiled to correspond to these blocks. The orthoimages are tiled to provide a standard engineering drawing at a scale of 1" = 100', and there are typically 7-10 of them per block. The surveys and photogrammetry were performed by private architectural and engineering (A&E) contractors for the Corps of Engineers. Each pool was awarded to a different contractor. As a result, there are inconsistencies in the data provided when one pool is compared to another.
Contours, spot elevations and a DTM in ARC/INFO and Intergraph format were provided for all pools. Additional coverages such as cross sections and design files of other features, such as utilities and building footprints were provided at the discretion of the individual contractors. The orthoimages are 8-bitgreyscale TIFF; pixel size is 1x1 foot. Individual images are 3000 x 2500 pixels, and 375 of these 7.5 Mb files are required to cover the flood plain: 146 for Pool A, ?? for Pool B, 114 for Pool C and ?? for Pool D.
Index contours occur at five foot intervals with supplemental contours at one foot intervals.
Photogrammetrically derived spot elevations are at approximate 60 foot grid spacing where the ground was not obscured.

Digital terrain models are in the form of ARC/INFO floating point Grids, Intergraph DTMs and TTNs with a 60 foot cell size.

Purpose:
The data were collected to support and monitor the progress of the KRR, an environmental restoration project designed to return hydrology in the flood plain to a state more closely resembling that which preceded the construction of C-38. The data are suitable for applications that require detailed elevation data and high resolution aerial orthoimagery of this area. Some examples of these are inundated area mapping, land use determination, and existence of structures, roads and drainage features on lands subject to inundation.

Time_Period_of_Content:
Time_Period_Information:
Single_Date/Time:
Calendar_Date: 19950401
Currentness_Reference: Publication date of sources

Status:
Progress: In work
Maintenance_and_Update_Frequency: None

Spatial_Domain:
Bounding_Coordinates:
West_Bounding_Coordinate: -81.2590
East_Bounding_Coordinate: -80.8370
North_Bounding_Coordinate: 27.8250
South_Bounding_Coordinate: 27.1090

Keywords:
Theme:

(Continued)

Table 8-2 (Concluded)

Theme_Keyword_Thesaurus: none
Theme_Keyword: Aerial photography
Theme_Keyword: Topography
Theme_Keyword: Orthoimagery
Theme_Keyword: DTM
Place_Keyword_Thesaurus: Geographic Names Information System
Place_Keyword: Florida
Place_Keyword: Kissimmee
Place_Keyword: C-38
Place_Keyword: Osceola County
Place_Keyword: Polk County
Place_Keyword: Okeechobee County
Place_Keyword: Highlands County
Access_Constraints: none
Use_Constraints:
These data were collected and processed for government use in a specific US Army Corps of Engineers (USACE) activity. The Jacksonville District makes no representation as to the suitability or accuracy of these data for any other purpose and disclaims any liability for errors that the data may contain. As such, it is only valid for its intended use within its content, time and accuracy specifications. While no explicit constraints are placed on the use of this data, please exercise appropriate and professional judgement in it's use and interpretation.
Metadata_Reference_Information:
Metadata_Date: 19950824
Metadata_Contact:
Chief, Survey Section, Engineering Division

Contact_Information:
Contact_Organization_Primary:
Contact_Organization:
  US Army Corps of Engineers
  Jacksonville District
  Survey Section, CESAJ-EN-DT

Contact_Address:
Address_Type: mailing address
Address: PO Box 4970
City: Jacksonville
State_or_Province: FL
Postal_Code: 32232-0019
Contact_Voice_Telephone: 904-232-1606
Metadata_Standard_Name: FGDC Content Standards for Digital Geospatial Metadata
Metadata_Standard_Version: 19940608

Chapter 9
Hardware, Software and Network Issues and Standards

9-1. General

Configuring and procuring the GD&S is a challenge, considering all the combinations of hardware, software and connectivity options available, but a solid set of requirements will allow purchases to be made with confidence. These requirements include the user needs, as defined in the systems requirements analysis, and non-user needs, the rules and regulations that must be followed during a system procurement, which may affect system design. GD&S design can be divided into hardware, software and network configuration.

9-2. Hardware

a. Compliance with standards. GD&S design must be done in phases and certain subjects must be dealt with in the early stages of system configuration. For instance, there are Federal standards that impose various standards on internetworking and interoperability among computer systems.

The United States Government has adopted the International Standard ISO/IEC 9945-1:1990, Information Technology - Portable Operating System Interface (POSIX) - Part 1: System Application Program Interface (API) [C Language}, as a Federal Information Processing Standard (FIPS 151-2, May 12, 1993). This standard defines a C programming language source interface to an operating system environment for scientific workstations, multi- user, and multi-tasking systems. This standard does not apply to the desktop or single user environment. This standard is for use by computing professionals involved in system and application software development and implementation. An objective of this standard is to promote portability of useful computer application programs at the source code level. FIPS 151-2 supersedes FIPS 151-1 and FIPS 151.

FIPS 146-2 (May 15, 1995), Profiles for Open Systems Internetworking Technologies (POSIT), removed the requirement that federal agencies specify Government Open Systems Interconnection Profile (GOSIP) protocols when acquiring computer networking products and services and communications systems or services.

b. Architecture. There are two basic system architectures in use, the client/server model and the stand-alone workstation model.

The client/server architecture allows all the large datasets and software packages to be stored on one central server system, freeing the other client machines to use their resources for other needs. This model provides better data management, allows for greater expansion due to the lower cost of typical client machines, which could be a simple X terminal, and provides single-point control of the databases and software used in the system. The server machine must have sufficient computing muscle and storage capacity to satisfy the needs of multiple users. This will drive up the cost of the initial server machine beyond what a typical stand-alone workstation would cost, but additional client machines will cost substantially less, particularly if that client machine is just an X terminal. The server can act solely as a file server for data storage and retrieval, or it can also provide computation services due to its fast CPU. In this aspect, the server could be viewed as a powerful standalone workstation with the capability to share its resources through a network.

The stand-alone system does not have shared resource requirements, so the performance of the CPU is not as important, but the system does require that every peripheral and upgrade path will be available during the lifetime of the GD&S. The purpose of a stand-alone GD&S could be to ensure strict security on the data since there are no channels for outside accesses. In addition, a stand-alone design can place networking as a low priority since it is rarely if ever used. This will severely limit the amount of data shared among GD&S and reduces the available resources the GD&S can use for scientific analysis.

c. Operating system (OS). The OS that is used must be compatible with the GD&S software and provide services to satisfy user needs and networking needs. In addition, consideration must be given to optional software and peripheral interfaces. The choice may be very simple if existing hardware has already been standardized on one OS; otherwise, the choices that are available are increasing and within three years the situation could change dramatically. The major OS choices available are UNIX (most are POSIX compliant), Windows, Windows NT (POSIX compliant), Windows95, DOS, VMS, and OS/2. Since the availability of these operating systems has grown over the past few years, the system designers choice of OS could be based on preference, but a shrewd choice can substantially benefit the GD&S and can prevent unnecessary costs when changes must be made. When choosing an OS for a time-critical GD&S, consider track record and dependability to be very important and avoid recent upgrades and opt for the older, more secure versions.

UNIX has been the overall standard in scientific computing over the last 15 years, mostly because of its wide availability over varying platforms and its close relationship with

developers. UNIX offers standard networking based on TCP/IP protocol which is available with every version of UNIX. In addition, multiple users and multiple concurrent processes are supported, which makes it a likely choice for multiple user systems. Each user on a UNIX based system has a user account and password, which allows the system to maintain security and generate accounting information for billing or resource management. Recently, UNIX has undergone a graphical facelift much like Windows for DOS, which may come from different vendors. Through the X Consortium's graphical server standard many graphical environments are offered to provide a graphical user interface to UNIX, including HP-VUE, Motif, OpenWindows and SunView. This OS has been around for many years and has a very respectable track record and offers many services that other operating systems do not.

VMS is the proprietary OS developed by Digital Equipment Corporation (DEC) to operate on their line of VAX machines and has the same basic capabilities as UNIX minus the wide availability with different hardware and platforms. OS/2 is developed by International Business Machines (IBM) and offers a fully 32 bit operating system architecture for PC class machines. In addition, it provides compatibility with Microsoft Windows and DOS. Microsoft offers Windows NT, a 32-bit operating system with advanced networking and security features and maintains the standard Windows interface. Microsoft also offers Windows and DOS operating systems for PC class machines, which provides access to thousands of commercial software packages with substantially lower cost than a typical UNIX based software package. Apple's proprietary OS for the Macintosh line of computers maintains a simple plug-and-play peripheral interface.

d. Compatibility with existing hardware. Integration of a new GD&S into an existing system must be done with care and foresight in order to reduce incompatibility problems. If the existing systems are linked via a network and the new GD&S is proposed to be integrated into the existing network, then make sure the networking protocols and transports are supported on all machines. Some systems allow multiple protocols and transports to be used, however it is much more stable and safe to use the same network protocols (e.g., TCP/IP) and transports (e.g., 10BaseT). Additional cost can be saved if peripherals can be shared or used on all machines within the GD&S, consequently, verify that the existing peripherals can be used by the new GD&S hardware and software. It is generally a good idea to limit the number of different vendors and manufacturers to as few as possible in order to reduce maintenance contract costs and hardware/ software conflicts that arise between products from different manufacturers. This is especially true with software since developers rarely have the time and resources to test their products on every hardware configuration.

e. Upgradeability. Considering the pace that technology increases - a hardware generation is approximately seventeen months - it is inevitable that hardware and software upgrades will be desirable to increase performance and offer more complex analytical functions. It is very important to choose hardware and software that can be upgraded in as many ways as possible. Determine if CPU upgrades, RAM expansions, video expansions and storage expansions are available immediately or in the near future. Currently, the more flexible mode of upgrade is to offer a few hardware "slots" that are connected to the CPU bus architecture so that upgrade boards can be added to the system, such as to provide accelerated video and faster networking throughput.

f. Peripheral support. A GD&S is composed of a CPU, GD&S software, georeferenced datasets and additional peripherals that provide input and output to the system. There are many peripherals that should be accessible through the GD&S, such as scanners, digitizers, backup devices, GPS post processing, printers and plotters. Peripherals are an integral part of a GD&S and must be supported.

g. Maintenance and Management. Hardware maintenance is a very important job that generally requires a staff devoted to this task. Large computer systems of the past require expensive maintenance contracts and require constant attention. The purchaser has either the option to purchase a maintenance contract from the vendor or develop a trained staff for maintaining the computer equipment. Both options are costly, but system health must be maintained in order to have a working GD&S. This is especially true in a network, where if the network was corrupted or failed, then GD&S work would come to a halt until the problem could be fixed. In these situations, having service staff available may save a project from failure.

A System Administrator position should be created to manage the GD&S hardware and network products. The System Administrator shall maintain the system and develop standard operating procedures for the GD&S, such as user accounts, project schedules, scheduled downtimes for maintenance and CPU utilization reports.

h. Example configurations.

(1) SUN Sparcstation 10.

(a) 40 MB RAM.

(b) (2) 1.2 GB hard drives.

(c) Keyboard.

(d) Mouse.

(e) Wacom digitizing tablet.

(f) CD-ROM drive.

(g) Exabyte backup drive.

(h) 17' 256 color monitor.

(2) Intergraph Microstation.

(a) Keyboard.

(b) CD-ROM drive.

(c) 20' color monitor.

(d) Digitizing tablet.

(3) 486 33Mhz PC-compatible.

(a) 8 MB RAM

(b) 500 MB hard drive.

(c) Mouse.

(d) Keyboard.

(e) Syquest 88MB removable hard drive.

(f) CD-ROM drive.

(g) 14' color monitor.

(h) Ethernet networking card.

(i) 256 color Super-VGA video card.

(j) 600 dpi monochrome laser printer.

9-3. Software

a. Compliance with standards. In order for a GD&S to operate efficiently, a dependable strategy must be designed to handle importing and exporting datasets of differing formats and platforms. The Spatial Data Transfer Standard (SDTS), FIPS 173, is a mechanism for the transfer of spatial data between dissimilar computer systems. The SDTS specifies exchange constructs, addressing formats, structure, and content for spatially referenced vector and raster data. The SDTS Fact Sheet by the U.S. Geological Survey states that the advantages of SDTS include data and cost sharing, flexibility, and improved quality, all with no loss of information. This standard is mandatory for Federal agencies and will serve as the spatial data transfer mechanism for all Federal agencies. The U.S. Geological Survey (USGS) has been designated as the maintenance authority and will be producing software tools and guidelines for SDTS data encoding and decoding. They will also offer SDTS workshops and training in order to educate the spatial data community. The GDS software chosen must support SDTS as one of its data import/export format options.

For further information regarding the SDTS (FIPS 173), contact:

U.S. Geological Survey
SDTS Task Force
526 National Center
Reston, VA 22092
FAX: (703) 648-4270
E-mail: sdts@usgs.gov

b. Export/import raster and vector data. The GDS software must have the capability to import a variety of commonly available data formats without loss of information. This includes vector and raster data and associated descriptive attribute (tabular) data. At a minimum, the GDS software should have routines for importing commonly available data from government and commercial sources. These formats include DEM and DLG (USGS), TIGER-Line (Census Bureau), DTED and VPF (DMA) and standard commercial image formats from SPOT and EOSAT. In addition, the software should accept the widely used DXF exchange format, SDTS formats and ascii data.

Data export formats supported by the GDS software can be more limited. Data exchange between GDS using the same software is generally easy. The SDTS governs the exchange of spatial data between dissimilar systems among Federal agencies.

DMA has developed several data formats as military standards. However, these military standards are not FIPS and the GDS software chosen is not required to read or write data in these formats. The Vector Product Format (VPF), MIL-STD-2407 is a standard vector data format used by the Defense Mapping Agency and many of their map products are distributed in that format. DMA has also developed the Raster Product Format, MIL-STD-2411, for image and scanned data in compressed or uncompressed form.

c. Visual environment. There are certain minimum visual requirements that the GD&S environment must fulfill so that data analysis can be properly done. The GD&S must be able to display the datasets in a georeferenced environment and preferably allow the user to zoom into and

away from the dataset based on certain spatial characteristics. Secondly, the user must be able to query the dataset either spatially or topically. Otherwise, the data remains strictly graphical and detailed analyses and reports can not be accurately performed. Third, the GD&S must provide the user with the capability to manipulate the dataset in some intelligent manner. Suppose the user must create a demonstration and the project calls for particular locations passing certain qualifications to be set to the color blue. The user will need to be able to manipulate the dataset as it is displayed to reflect these requirements. Data integrity should not be violated, but a certain latitude must be given to the user so that the environment can be as flexible and usable as possible.

d. Maintenance and management. GD&S maintenance requires a standard operating procedure for data archiving, data security and data modification. A typical data archiving schedule consists of daily data backups and weekly full system backups. Archiving data will ensure that any corruption or system failure will not destroy important data or software. Data security measures such as user and project login with password entry to the GD&S are standard. In a distributed GD&S network, data stewardship can be allocated such that data management and security can be centralized in one organization. However, a stand-alone GD&S workstation will require more control and might require strict passwords and possibly even read-only access to critical datasets. Computer systems are not fail safe and as many provisions that can be taken to prevent data loss or data corruption must be made. If datasets must be edited or updated, then a standard operating procedure should be drawn up for such an occasion, and a version control procedure should be implemented. Full backups of each successive data state should be kept in case the decision is made to reverse the changes.

9-4. Networking

a. Overview. In the early days of the personal computer revolution, people were content to use their computers for personal finance and possibly a little word processing. However, as hardware and software capabilities grew, they realized that these computers could offer them much more. Today's high-powered systems are used in all facets of business, including the quickly- growing GDS market. A computer is simply much better at GDS activities than people performing them by hand because it can analyze multiple multi-record databases and correlate an information base into useful results quick enough for them to be utilized in the real world. For example, the Federal Emergency Management Agency (FEMA) now uses a GDS to manage natural and man- made disasters. This GDS allows the agency to track hurricanes, estimate damage through simulations, and plan relief efforts in a matter of minutes; FEMA can now better manage relief aid for such disasters and then use the data gathered during one hurricane to plan for the next. Applications of GDS like this save time and lives, but are dependent upon the availability of large and often costly databases. Databases can quickly surpass your system's on-line storage capacity, requiring access to (and storage on) other machines, both local and remote. You may also have a need to send output from your GDS system to customers through quicker means than the postal service. Networking was born some twenty years ago to solve such problems. At that time, visionaries had an idea of computers becoming information resources for any number of different subjects. This idea has become a reality with the advent of the Internet. Once you are connected to the Internet, you have instant access to a wealth of information. Through electronic mail and bulletin boards (called "news groups" in Internet lingo), you can use a different kind of resource: a worldwide supply of knowledgeable people, their software, and their hardware.

This section provides a *brief* introduction to the subject of networking. This information could easily fill a book in its own right. The Internet, as described in this document, is still evolving along with the means of accessing it. Therefore, it should be known that the information presented in this document is current as of early August 1994 and may change within the next several years.

b. Introduction to networking and related concepts. Networking, simply put, is connecting your computers together so they can share information. Effective networking increases productivity by using computer resources, such as files, printers, and memory, more efficiently. A network puts the power of all your system's hardware and software at your fingertips.

Although there are many different types of networks, they fall into two general categories. Local area networks (LAN) are small groups of computers in close proximity connected together on a single line. Wide area networks (WAN), such as the Internet (although the Internet is really a collection of WAN, it can be viewed here as one large network) connect computers that can be as close as several hundred feet to as far as across the globe and typically include connections via cable, telephone lines, and satellites.

A network, in the physical sense, consists of cables or phone lines which directly connect computers, and special hardware installed in each computer which provide the means for communication. Computers on a network have established ways of communicating, called protocols. Protocols dictate which signals computers use across cables, how they tell one another that they have received information, and how they exchange information. These protocols are broken down into layers of service which

specify the hardware/software interactions taking place on the network. Each layer is responsible for one piece of the communication and interacts only with the layers above and below it. For example, the International Organization for Standards (ISO) has a seven-layer protocol called the Reference Model for Open Systems Interconnection (OSI), as shown in Table 9-1 (lower layers are closer to hardware and higher layers are closer to the user).

A basic networking package consists of several services that allow your computer to connect to both local and wide area networks.

TCP/IP provides various protocols for networking communications that other networking packages, such as NFS, use. The TCP/IP package also provides end-user programs such as telnet and ftp that enable remote login and file transfer between systems running TCP/IP.

NFS allows you to export file systems to the network so that users on other computers can use them as if they were local, and to import file systems from remote systems to your own.

The Multichannel Memorandum Distribution Facility (MMDF) controls electronic mail communications between machines. MMDF is a service that provides users with transparent access to different networks and related mail transport protocols.

c. Local area networks.

(1) Description of a LAN. A local area network is a direct connection of computers in the same office or in adjacent buildings on a single cable. Within this small circle, each computer can communicate with every other computer on the LAN and share resources such as hard drives, printers, and any other peripherals. A LAN typically has one or more machines which perform services on behalf of the other computers in the LAN. These servers usually perform printing and file storage services. In addition, a server is also used to route connections to the Internet and other LANs around the world. The USACE policy on LANs is stated in IM Policy Memo 25-1-8, "Local Area Network (LAN) Design and Interoperability."

(2) NFS/NIS. When the TCP/IP protocol was first developed and implemented on UNIX workstations, it was wonderful to be able to retrieve specific data from other machines on your LAN through such programs as ftp. However, with larger programs being developed, this became impractical. For example, most GDS systems require a large amount of online data which can quickly consume computer disk space. Instead of requiring each machine on the LAN to have its own copy of such data, it would be better to have one central copy on a server which everyone can access without having to transfer it to their machine. FTP does not provide the capabilities necessary to perform this task. In the early 80's Sun Microsystems developed a service called NFS (Network File System) which revolutionized the computer industry. NFS is a set of protocols which allow your computer to use files on other systems as if they were local. So, instead of obtaining files via ftp, you can read, write and edit files on remote machines. NIS is the service which manages all NFS activities. Its primary purpose is to ensure network consistency and maintain file access security in a LAN. NIS accomplishes this task by maintaining lists of users, passwords, and system access availability, and distributing the lists throughout the LAN. Each machine then knows who has access to specific files.

(3) Hardware and software issues. When the only computers you could buy included the UNIX operating system, building a LAN was not a problem because the basic architecture of each system's UNIX was the same. So, even if you had UNIX computers from two different vendors, you could be certain that the two machines would communicate without any problems on your LAN. Times have changed, and there are now many different operating systems available for every computer you can buy. Not all of them come with TCP/IP software. For example, while UNIX comes with such a package, MSDOS requires extra add-on software. Even with TCP/IP installed on both platforms, you still might not be able to share data due to physical differences between the machines such as the way one computer stores binary information as opposed to the other. The point is that you must put some thought into building your LAN. Make sure that all the different types of computers you wish to include will work cooperatively together or that you can purchase software to make them work together. Sun sells software which allows PCs to communicate with UNIX machines as do other vendors. Macintosh computers come ready with TCP/IP, but do not automatically support NFS. In addition, you should thoroughly investigate any hardware limitations you might encounter. If you plan out a strategy for purchasing equipment and computers for your LAN ahead of time, it will save you a world of data access and transfer nightmares.

d. The Internet. The Internet is a worldwide collection of WANs linking computers from government, academia, and industry. It allows easy access for computer users, and so is being used by the FGDC for the establishment of the National Geospatial Data Clearinghouse.

(1) History of the Internet. The Internet began as a Department of Defense experiment to develop a computer network which could operate without data loss in a battle during wartime when partial outages are likely. Because of

the premise on which it was developed, the ARPAnet (as the Internet was originally called), placed all the demands of data transmission and decoding on the network computers rather then the actual network links. This meant that the network did not have a vulnerable main transmission path or decoding hub. If one link vanished, the network computers determined this and rerouted their data through another path.

While this experiment was taking place, universities and other government agencies, who had most of the computing power in the United States, were looking for ways to link their systems together for research and educational purposes. However, since these organizations had no policies regarding computer purchasing, they found it difficult to successfully link computers from different manufacturers. The ARPAnet development solved this problem because of its basis on computers transmitting and decoding data themselves. If you wanted your computer to have access to other types of computers, you only had to write a piece of software for your computer which could send and receive data using the Internet Protocol (the method by which data is transmitted on the Internet). Demand to connect university and government mainframes grew, and Internet developers, responding to pressures, began to develop their Internet Protocol software on every possible type of computer. It quickly became the only practical way for computers from different manufacturers to communicate.

Roughly 10 years later, Ethernet local area networks (LAN) and workstations appeared on the scene. Most of them came with Berkeley UNIX, which came with the necessary Internet Protocol software built-in (this is where the fallacy that the Internet is UNIX started - although most of the Internet Protocol software originated on UNIX platforms, it is not part of the UNIX operating system itself). A new demand was created not only to connect large time-sharing mainframes to the ARPAnet but also to connect the entire LAN. Many different organizations started to build their own networks based on the Internet protocols because they found the existing ARPAnet communications too limiting due to its bureaucracy and staffing problems. Probably the most important of these new networks was the National Science Foundation's (NSF) NSFNET. In the 80's, the NSF established five centers for supercomputing activities. The fastest computers had previously been available only to the military and researchers from the government and universities. The NSF was now making such resources available to any world-wide researcher. Only five centers were established because of costs. To connect each center they needed dedicated, high-speed phone lines which were paid for by the mile. They realized that connecting each university to a center would bankrupt the agency, so they established regional networks with links to the supercomputing centers. Each university was connected to its nearest neighbor, and each computer in the network chain would forward requests between computers in the chain. This solution was very successful and has been adopted for modeling current networking activities.

In the late 80's, with network traffic increasing by about 15 percent per year, the Internet had grown to the point that its routing computers and the telephone lines connecting them were overloaded. The Federal government invested large sums of money into upgrading the entire network, and it continues to be the driving force behind supporting the Internet. These upgrades included new telephone lines with higher data transfer rates and more powerful data routing systems. With these enhancements to the Internet, it has become possible for more people to use the Internet not only in government, universities, and corporations, but also in their own homes. Various national companies, such as DELPHI, now offer Internet connections through standard modems. This type of connection may be slower than those available to organizations, but it is much more reliable than earlier bulletin board services to which the average home user had access. In addition, through such a connection, you have access to any computer on the Internet and a multitude of data and services.

(2) Internet security issues. Since the Internet is a global community of networks, anyone can have access to your system at any time. USACE Commands must control the types of access by establishing some basic security procedures before placing systems on the network. Each system administrator must be aware of the risks and take appropriate action to protect USACE systems without limiting access to the services available in Internet. USACE requirements for Internet Security are described in EP 25-1-97, Internet Implementing Procedures.

e. Obtaining Information from the Internet. This section will give a brief overview of services and applications which are used to access the Internet. Appendix C lists specific Internet sites that provide GD&S information.

(1) Telnet. Telnet is used for logging into other computers on the Internet or even to other computers on your LAN. Once you have established a connection, your machine is operating as a dummy terminal to display messages from the remote machine. It is used primarily for text-based purposes such as searching library card catalogs and other kinds of databases. Telnet generally does not offer file transfer capabilities between connected machines other than simple ASCII text.

Table 9-1
Reference Model for Open Systems Interconnection

Layer	Name	Description
7	Application	Defines how programs can communicate with each other.
6	Presentation	Performs any necessary data conversion.
5	Session	Establishes an enhanced connection (session) with other machines.
4	Transport	Makes sure information exchanged between computers arrives intact and without errors.
3	Network	Makes sure information coming from one computer arrives at the correct destination.
2	Data Link	Makes sure information gets from one end of the cable to the other intact.
1	Physical	Defines the way information travels on a cable. A common physical-level protocol is the IEEE 802.3 protocol, of which Ethernet is a subset.

(2) FTP. FTP (File Transfer Protocol) allows you to move files between machines. Once a connection is established, you may view the directory structure of the remote machine and move files to/from that computer, but you may not run any applications on that system. FTP is most useful for retrieving files from public archives that are scattered on the Internet. This is called an "anonymous FTP" because you do not need an account to access the remote computer.

(3) Electronic mail. Electronic mail, or e-mail, lets you send and receive messages to/from a person at a remote location on the Internet. E-mail is usually used for quick informal conversing over the Internet and as a means to quickly distribute textual information to multiple parties. Present E-mail software can even distribute embedded documents from popular word processors and other files.

(4) Network news. USENET is a system that lets you read (and post) messages that have been sent to public "news groups" (news groups are the equivalent of bulletin boards in the Internet community.) It is used primarily to converse with other people on any subject. For example, if you had a question about a problem with your laser printer, you could post a message to this service asking if anyone has a solution. USENET is the world's largest bulletin board service.

(5) Gopher clients. Gopher is a tool for browsing the Internet by selecting resources from a menu. Most often you will view the resources available in a directory tree structure. Gopher client software is designed to be user friendly so that it can help you get access to a particular item of interest. You do not have to worry about Internet addresses and once it has established a connection for you, the software can even retrieve files. Because of this, gopher clients are like ftp software packages with a user-friendly front-end. To use this type of package, the remote machine must be configured as a gopher server. This service is relatively new in the Internet community, so you will mainly find gopher servers at universities.

(6) WAIS. Wide Area Information Servers (WAIS) is another new service available on the Internet and is being used by FGDC. It is good for searching through indexed material and finding articles based on what they contain. WAIS is really a tool for working with large collections of data and databases. Almost any type of information can be indexed, and the retrieval software allows natural language string searches. For example, you could use WAIS to search indexed GDS databases for all references to Lake Michigan. In addition to searching for information in documents and databases, WAIS also has the capability to show you the information itself.

(7) The World Wide Web. The World Wide Web (WWW) is the newest information service to arrive on the Internet. This service is based on a technology called hyperlinking. Hyperlinking is a means of providing point-and-click access to other document sources - text, image, or sound - which relate to what you are currently viewing. Anyone who has used the Microsoft Windows Help program has seen this technology at work. Sections of text can have links to other sections through keywords. These keywords are highlighted via underlining, italics, etc. When you click on one of these keywords with a mouse, the software "fetches" the document corresponding to that link. The Web operates in the same way, but it goes much further. The links in the Web can be either text-based, pointing to other Internet sites and particular files which you may download to your machine, or multimedia pictures and audio which may be played on your computer. The purpose of the Web is to provide a subject-oriented means of browsing the Internet, which makes finding particular files and information much easier. Instead of locating files through text based searches with ftp and telnet, you are presented

with a paragraph on a particular file with a direct link to that file somewhere on the Internet.

Since the Web is relatively new, not many hyperlink Internet sites exist. These sites are identified by the letters "http" at the beginning of their address. In addition, it is up to the owners of each Internet site to determine whether or not to build hyperlink pages for their computers. If you are planning to let the public use your computer, it is a much more user friendly way to provide access to your services and resources. From a user's point of view, these hyperlink documents would be useless if there were no way to view them. Fortunately, the Web has been strongly endorsed by the Internet community, and there is an effort under way to develop such applications. Mosaic, which is being developed by the National Center for Supercomputing Applications, is the most popular of these hyperlink Internet browsing tools. This software offers all the features of the Web plus capabilities for WAIS, ftp, telnet, and gopher in one package, and is being developed for UNIX workstations running X-Windows, Macintoshes, and PCs with Microsoft Windows. The latest version of Mosaic is available free on the Internet with new alpha releases being distributed for all three environments about once every month. For further information on Mosaic, contact:

NCSA Documentation Orders
152 Computing Applications Building
605 East Springfield Avenue
Champaign, IL 61820-5518
(217) 244-4130
orders@ncsa.uiuc.edu

Chapter 10
System Procurement

10-1. General

a. The term "Information Technology (IT)" is defined in Public Law (PL) 104-106, Section 5002 Definitions, (3) (b). IT includes computers, ancillary equipment, software, firmware and similar procedures, services (including support services) and related resources.

b. The passage of PL 104-106 has shifted the responsibility for management and oversight of IT from the General Services Administration (GSA) to the Office of Management and Budget (OMB). GSA abolished the FIRMR effective 8 Aug 96. OMB is required to issue guidance in conducting IT acquisitions. Your local Director or Chief of Information Management will be able to provide you with the latest guidance and direction in defining and developing the appropriate documentation to justify initiating the acquisition process. As a minimum, requirements should have been identified in your organization's IMA Mod Plan and the IT assets captured in the Requirement Statements Management System (RSMS). If solicitation or contract does not require IT then a brief statement must accompany the request that states, the specification for this contract do not contain any requirement for IT."

c. The documentation required to justify initiating the acquisition process is, generally: stating a specific mission that needs IT resources to satisfy that mission, with measurable benefits derived from the investment. All of these must be consistent with common sense and sound business practices.

(1) The planning for IT resource requirements starts with establishing the mission need. The needs identified at program initiation must be reexamined at each milestone to assure that they reflect the most current program conditions and IT. The following are major elements in the acquisition process: mission needs, structuring an acquisition strategy, developing producible and affordable designs, making decisions, and assessing program status as it applies to Life Cycle Management of Information System (LCMIS).

(2) A part of the requirement justification must identify the IT resource being requested, in order that the Director or Chief of Information Management can certify compliance that it is consistent with Army Technical Architecture. A statement justifying the requirement is required, and a financial analysis under LCMIS may be required. You need to assure that requirement statement has been updated and still conveys the justification for the IT resource.

10-2. Contract Vehicles

This section lists the points of contact for some existing contract vehicles that can be used by USACE Commands to procure GDS hardware and/or software. These contracts are applicable to GDS procurements but are by no means the only vehicles available. Using an existing contract vehicle eliminates some of the procurement steps. However, all hardware procurements must have approved Life Cycle Management documentation, a Requirements Analysis and Analysis of Alternatives.

The most important contract for acquisition of GDS hardware and software is the "Facilities CAD2" contract. The CAD2 contract was awarded to two vendors, Intergraph (Contract N66032-93-D-0021) and Cordant, Inc. (Contract N66032-93-D-0022). Both vendors offer a wide variety of software and hardware products and support services for GD&S applications.

The USACE point of contact for the Facilities CAD2 contract is:

Mr. C. W. "Rusty" Brasfeild, Jr.
Facilities CAD2 COR
NAVFAC DET WES, Bldg 8000
3909 Halls Ferry Road
Vicksburg, MS 39180-6199
Telephone: (800) 700-2232 or (601) 634-4474
FAX: (601) 634-2947

GDS software and hardware are also available from the CEAP-IA. This contract supports the Corps of Engineers Automation Plan (CEAP) and is an indefinite delivery, indefinite quantity contract. The ESRI line of ARC/Info GDS software products are available from this contract. It also provides multiuser SUN and Control Data 4000 workstations, SUN and Control Data 4000 UNIX minicomputers, Control Data CYBER mainframes, local (LAN) and wide area network (WAN) hardware and software, the ORACLE database management system with associated products, and professional support services.

The USACE Point of Contact for the CEAP-IA is:
Mr. Ken Calabrese
Headquarters, U.S. Army Corps of Engineers
ATTN: CEIM-S (Ken Calabrese)
20 Massachusetts Avenue, N.W.
Washington, D.C. 20314-1000
Telephone: (202) 761-1244

The Small Multi-user Computer-II (SMC-II) contract was awarded to Telos and supplies peripherals, software, networking components, maintenance, and engineering

services. The USACE Point of Contact for the SMC-II contract:

Ms Adelia Wardle
PMAIS/USAISMA
ATTN: ASQM-SWM
Bldg. 283
Fort Monmouth, NJ 07703-5605
Telephone: (908) 532-7944
Internet: wardlea@isma8.monmouth.army.mil

Chapter 11
ACCURACY STANDARDS FOR ENGINEERING, CONSTRUCTION, AND FACILITY MANAGEMENT SURVEYING AND MAPPING

11-1. Scope. This chapter provides technical guidance on engineering surveying and mapping accuracy standards used in engineering and construction. It is intended for use in developing specifications for geospatial data used in various project documents, such as architectural and engineering drawings, master planning maps, construction plans, navigation project condition charts and reports, and related GIS, CADD, and AM/FM products. Guidance is provided for preparing specifications for surveying and mapping services.

11-2. General Surveying and Mapping Specifications. Construction plans, maps, facility plans, and CADD/GIS data bases are created by a variety of terrestrial, satellite, acoustic, or aerial mapping techniques that acquire planimetric, topographic, hydrographic, or feature attribute data. Specifications for obtaining these data should be "performance-based" and not overly prescriptive or process oriented. They should be derived from the functional project requirements and use recognized industry accuracy standards where available.

a. Industry standards. Maximum use should be made of industry standards and consensus standards established by private voluntary standards bodies; in lieu of Government-developed standards. Therefore, industry-developed accuracy standards should be given preference over Government standards. A number of professional associations have published surveying and mapping accuracy standards, such as the American Society for Photogrammetry and Remote Sensing (ASPRS), the American Society of Civil Engineers (ASCE), the American Congress on Surveying and Mapping (ACSM), and the American Land Title Association (ALTA). When industry standards are non-existent, inappropriate, or do not meet a project's functional requirement, FGDC, DoD, Tri-Service, DA, or USACE standards may be specified as criteria sources. Minimum technical standards established by state boards of registration, especially on projects requiring licensed surveyors or mappers, should be followed when legally applicable. Local surveying and mapping standards should not be developed where consensus industry standards or DoD/DA standards exist.

b. Performance specifications. Performance-oriented (i.e., outcome based) specifications are recommended in procuring surveying and mapping services. Performance specifications set forth the end results to be achieved (i.e., final map format, data content, and/or accuracy standard) and not the means, or technical procedures, used to achieve those results. Performance-oriented specifications typically provide the most flexibility and use of state-of-the-art instrumentation and techniques. Performance specifications should succinctly define only the basic mapping requirements that will be used to verify conformance with the specified criteria, e.g., mapping limits, feature location and attribute requirements, scale, contour interval, map format, sheet layout, and final data transmittal, archiving or storage requirements, required accuracy criteria standards for topographic and planimetric features that are to be depicted, and quality assurance procedures. Performance-oriented specifications should be free from unnecessary equipment, personnel, instrumentation, procedural, or material limitations; except as needed to establish comparative cost estimates for negotiated services.

c. Prescriptive (procedural) specifications. Use of prescriptive specifications should be kept to a minimum, and called for only on highly specialized or critical projects where only one prescribed technical method is appropriate or practical to perform the work. Prescriptive specifications typically require specific field instrumentation, equipment, personnel, office technical production procedures, or rigid project phasing with on-going design or construction. Prescriptive specifications may, depending on the expertise of the writer, reduce flexibility, efficiency, and risk, and can adversely impact project costs if antiquated methods or instrumentation are required. Prescriptive specifications also tend to shift most liability to the Government. Occasionally, prescriptive specifications may be applicable to Corps projects involving specialized work not routinely performed by private surveying and mapping firms, e.g., mapping tactical operation sites, mapping hazardous, toxic, and radioactive waste (HTRW) clean-up sites, military/tactical surveying, or structural deformation monitoring of locks, dams, and other flood control structures.

d. Tri-Service CADD/GIS Technology Center standards. Tri-Service standards should be specified for in-house or A-E services requiring delivery of CADD, GIS, and other spatial and

geospatial data covered by this chapter--see Chapter 7 for detailed reference requirements.

e. Quality control. Quality control (QC) of contracted surveying and mapping work should generally be performed by the contractor. Therefore, USACE quality assurance (QA) and testing functions should be focused on whether the contractor meets the required performance specification (e.g., accuracy standard), and not the intermediate surveying, mapping, and compilation steps performed by the contractor. The contractor's internal QC will normally include independent tests which may be periodically reviewed by the Government. Government-performed (or monitored) field testing of map accuracies is an optional QA requirement, and should be performed when technically and economically justified, as determined by the ultimate project function.

f. Metrication. Surveying and mapping performed for design and construction should be recorded and plotted in the units prescribed for the project by the requesting Command or project sponsor. During transition to the metric system, inch-pound (IP) units or soft conversions may be required for some geospatial data.

g. Spatial coordinate reference systems. Where practical and feasible, civil and military projects should be adequately referenced to nationwide or worldwide coordinate systems directly derived from, or indirectly connected to, Global Positioning System (GPS) satellite observations. In addition, navigation and flood control projects in tidal areas should be vertically referenced to the latest datum epoch established by the Department of Commerce.

11-3. <u>Accuracy Standards for Engineering and Construction Surveying</u>. Engineering and construction surveys are performed to locate, align, and stake out construction for civil and military projects, e.g., buildings, utilities, roadways, runways, flood control and navigation projects, training ranges, etc. Engineering surveys are performed to provide the base horizontal and vertical control used for area mapping, GIS development, preliminary planning studies, detailed site plan drawings for construction plans, construction measurement and payment, preparing as-built drawings, installation master planning mapping, and future maintenance and repair activities. Most engineering surveying standards currently used are based on local practice, or may be contained in State minimum technical standards.

a. Accuracy standards. Engineering and construction surveys are normally specified and classified based on the horizontal (linear) point closure ratio or a vertical elevation difference closure standard. This type of performance criteria is most commonly specified in Federal agency, state, and local surveying standards, and should be followed and specified by USACE commands. These standards are applicable to most types of engineering and construction survey equipment and practices (e.g., total station traverses, differential GPS, differential spirit leveling). These accuracy standards are summarized in the following tables.

Table 11-1
Minimum Closure Accuracy Standards for Engineering and Construction Surveys

USACE Classification	Closure Standard	
Engr & Const Control	Distance (Ratio)	Angle (Secs)
First-Order	1:100,000	$2\sqrt{N}$[1]
Second-Order, Class I	1:50,000	$3\sqrt{N}$
Second-Order, Class II	1:20,000	$5\sqrt{N}$
Third-Order, Class I	1:10,000	$10\sqrt{N}$
Third-Order, Class II	1: 5,000	$20\sqrt{N}$
Engineering Construction (Fourth-Order)	1: 2,500	$60\sqrt{N}$

[1] N = Number of angle stations

Table 11-2
Minimum Elevation Closure Accuracy Standards for Engineering and Construction Surveys

USACE Classification	Elevation Closure Standard (ft)[1]	(mm)
First-Order, Class I	$0.013\sqrt{M}$	$3\sqrt{K}$
First-Order, Class II	$0.017\sqrt{M}$	$4\sqrt{K}$
Second-Order, Class I	$0.025\sqrt{M}$	$6\sqrt{K}$
Second-Order, Class II	$0.035\sqrt{M}$	$8\sqrt{K}$
Third-Order	$0.050\sqrt{M}$	$12\sqrt{K}$
Construction Layout	$0.100\sqrt{M}$	$24\sqrt{K}$

[1] $\sqrt{M}$ or $\sqrt{K}$ = square root of distance in Miles or Kilometers

b. Survey closure standards. Survey closure standards listed in Tables 11-1 and 11-2 should be used as a basis for classifying, standardizing, and evaluating survey work. The point and angular closures (i.e., traverse misclosures) relate to the relative accuracy derived from a particular survey. This relative accuracy (or, more correctly, precision) is estimated based on internal closure checks of a traverse survey run through the local project, map, land tract, or construction site. Relative survey accuracy estimates are always expressed as ratios of the traverse/loop closure to the total length of the survey (e.g., 1:10,000).

(1) Horizontal closure standard. The horizontal point closure ratio is determined by dividing the linear distance misclosure of the survey into the overall circuit length of a traverse, loop, or network line/circuit. When independent directions or angles are observed, as on a conventional traverse or closed loop survey, these angular misclosures should be distributed (balanced) before assessing positional misclosure. In cases where differential GPS vectors are measured in three-dimensional geocentric coordinates, then the horizontal component of position misclosure is assessed relative to Table 11-1.

(2) Vertical control standards. The vertical accuracy of a survey is determined by the elevation misclosure within a level

section or level loop. For conventional differential or trigonometric leveling, section or loop misclosures (in millimeters or feet) should not exceed the limits shown in Table 11-2, where the line or circuit length is measured in the applicable units. Fourth-Order accuracies are intended for construction layout grading work.

c. Geospatial Positioning Accuracy Standards. Many control surveys are now being efficiently and accurately performed using radial (spur) techniques--e.g., single line vectors from electronic total stations or kinematic differential GPS to monumented control points, topographic feature points, property corners, etc. Since these surveys may not always result in loop closures (i.e., closed traverse) alternative specifications for these techniques must be allowed. This is usually done by specifying a radial positional accuracy requirement. The required positional accuracy may be estimated based on the accuracy of the fixed reference point, instrument, and techniques used. Ratio closure standards in Tables 11-1 and 11-2 may slowly decline as more use is made of nation-wide augmented differential GPS positioning and electronic total station survey methods. The FGDC is currently (1998) proposing use of positional accuracy tolerances in order to more easily correlate surveying standards with mapping or geospatial positional accuracy standards--e.g., NSSDA.

(1) GPS satellite positioning technology allows development of map features to varying levels of accuracy, depending on the type of equipment and procedures employed. Given this rapidly developing technology, GPS surveying specifications rapidly become obsolete; therefore, it is best to follow GPS manufacturer's recommended procedures. The advent of Government and commercial augmented GPS systems allows direct, near real-time positioning of static AM/FM type features and dynamic platforms (survey vessels, aircraft, etc.). Site plan drawings, photogrammetric control, and related GIS features can be directly constructed from GPS or differential GPS observations, at accuracies ranging from 1 cm to 100 meters (95%).

(2) Accuracy classifications of maps and related GIS data developed by GPS methods can be estimated based on the GPS positioning technique employed. Permanent GPS reference stations (Continuously Operating Reference Stations or CORS) can provide decimeter and even centimeter-level point positioning accuracies

over wide ranges; thus providing direct map/feature point positioning without need for preliminary control surveys.

d. Higher-order surveys. Requirements for relative line accuracies exceeding 1:50,000 are rare for most facility engineering, construction, or mapping applications. Surveys requiring accuracies of First-Order (1:100,000) or better should be performed using FGDC geodetic standards and specifications. These surveys must be adjusted and/or evaluated by the National Geodetic Survey (NGS) if official certification relative to the national network is required.

e. Instrumentation and field observing criteria. In accordance with the policy to use performance-based standards, rigid prescriptive requirements for survey equipment, instruments, or operating procedures are discouraged. Survey alignment, orientation, and observing criteria should rarely be rigidly specified; however, general guidance regarding limits on numbers of traverse stations, minimum traverse course lengths, auxiliary azimuth connections, etc., may be provided for information. For some highly specialized work, such as dam monitoring surveys, technical specifications may prescribe that a general type of instrument system be employed, along with any unique operating, calibration, or recordation requirements.

f. Connections to existing control. Surveys should normally be connected to existing local control or project control monuments/benchmarks. These existing points may be those of any Federal (including Corps project control), State, local, or private agency. Ties to local Corps or installation project control and boundary monuments are absolutely essential and critical to design, construction, and real estate. In order to minimize scale or orientation errors, at least two existing monuments should be connected. It is recommended that Corps surveys be connected with one or more stations on the National Spatial Reference System (NSRS), when practicable and feasible. Connections with local project control that has previously been connected to the NSRS is normally adequate in most cases. Connections with the NSRS shall be subordinate to the requirements for connections with local/project control.

g. Survey computations, adjustments and quality control/assurance. Survey computations, adjustments, and quality control should be performed by the organization responsible for the actual field survey. Contract compliance assessment of a

survey should be based on the prescribed point closure standards of internal loops, not on closures with external networks of unknown accuracy. In cases where internal loops are not observed, then assessment must be based on external closures. Specifications should not require closure accuracy standards in excess of those required for the project, regardless of the accuracy capabilities of the survey equipment. Least squares adjustment methods should be optional for 2nd or lower order survey work. Professional contractors should not be restricted to rigid computational methods, software, or recording forms. Use of commercial software adjustment packages is strongly recommended.

h. Data recording and archiving. Field survey data may be recorded and submitted either manually or electronically. Manual recordation should follow standard industry practice, using field book formats outlined in various technical manuals.

11-4. <u>Accuracy Standards for Maps and Related Geospatial Products</u>. Map accuracies are defined by the positional accuracy of a particular graphical or spatial features depicted. A map accuracy standard classifies a map as statistically meeting a certain level of accuracy. For most engineering projects, the desired accuracy is stated in the specifications, usually based on the final development scale of the map--both the horizontal "target" scale and vertical relief (specified contour interval or digital elevation model). Often, however, in developing engineering plans, spatial data bases may be developed from a variety of existing source data products, each with differing accuracies--e.g., mixing 1 in = 60 ft topo plans with 1 in = 400 ft reconnaissance topo mapping. Defining an "accuracy standard" for such a mixed data bases is difficult and requires retention of the source of each data feature in the base. In such cases the developer must estimate the accuracy of the mapped features.

a. ASPRS Standard. For site mapping of new engineering or planning projects, there are a number of industry and Federal mapping standards that may be referenced in contract specifications. The recommended standard for facility engineering is the ASPRS "Accuracy Standards for Large-Scale Maps" (ASPRS 1990). This standard, like most other mapping standards, defines map accuracy by comparing the mapped location of selected well-defined points to their "true" location, as determined by a more accurate, independent field survey. Alternately, when no independent check is feasible or practicable, a map's accuracy

may be estimated based on the accuracy of the technique used to locate mapped features--e.g., photogrammetry, GPS, total station, plane table, The ASPRS standard has application to different types of mapping, ranging from wide-area, small-scale, GIS mapping to large-scale construction site plans. It is applicable to all types of horizontal and vertical geospatial mapping derived from conventional topographic surveying or photogrammetric surveys. This standard may be specified for detailed construction site plans that are developed using conventional ground topographic surveying techniques (i.e., electronic total stations, plane tables, kinematic GPS). The ASPRS standard is especially applicable to site plan development work involving mapping scales larger than 1:20,000 (1 in. = 1,667 ft); it therefore applies to the more typical engineering map scales in the 1:240 (1 in. = 20 ft) to 1:4,8000 (1 in. = 400 ft) range. Its primary advantage over other standards is that it contains more definitive statistical map testing criteria, which, from a contract administration standpoint, is desirable. Using the guidance in Tables 11-3 and 11-4, specifications for site plans need only indicate the ASPRS map class, target scale, and contour interval.

b. Horizontal (planimetric) accuracy criteria. The ASPRS planimetric standard compares the root mean square error (RMSE) of the average of the squared discrepancies, or differences in coordinate values between the map and an independent topographic ground survey of higher accuracy (i.e., check survey). The "limiting RMSE" is defined in terms of meters (feet) at the ground scale rather than in millimeters (inches) at the target map scale. This results in a linear relationship between RMSE and target map scale; as map scale decreases, the RMSE increases linearly. The RMSE is the cumulative result of all errors including those introduced by the processes of ground control surveys, map compilation, and final extraction of ground dimensions from the target map. The limiting RMSE's shown in Table 11-3 are the maximum permissible RMSE's established by the ASPRS standard. These ASPRS limits of accuracy apply to well-defined map test points only--and only at the specified map scale.

c. Vertical (topographic) accuracy criteria. Vertical accuracy has traditionally been, and currently still is, defined relative to the required contour interval for a map. In cases where digital elevation models (DEM) or digital terrain models (DTM) are being generated, an equivalent contour interval can be specified, based on the required digital point/spot elevation

accuracy. The contours themselves may be later generated from a DEM using computer software routines. The ASPRS vertical standard also uses the RMSE statistic, but only for well-defined features between contours containing interpretative elevations, or spot elevation points. The limiting RMSE for Class 1 contours is one-third of the contour interval. Testing for vertical map compliance is also performed by independent, equal, or higher accuracy ground survey methods, such as differential leveling. Table 11-4 summarizes the limiting vertical RMSE for well-defined points, as checked by independent surveys at the full (ground) scale of the map.

Table 11-3a. ASPRS Planimetric Feature Coordinate Accuracy Requirement (Ground X or Y in Meters) for Well-Defined Points

Target Map Scale	ASPRS Limiting RMSE in X or Y (Meters)		
Ratio m/m	Class 1	Class 2	Class 3
1:50	0.0125	0.025	0.038
1:100	0.025	0.05	0.075
1:200	0.050	0.10	0.15
1:500	0.125	0.25	0.375
1:1,000	0.25	0.50	0.75
1:2,000	0.50	1.00	1.5
1:2,500	0.63	1.25	1.9
1:4,000	1.0	2.0	3.0
1:5,000	1.25	2.5	3.75
1:8,000	2.0	4.0	6.0
1:10,000	2.5	5.0	7.5
1:16,000	4.0	8.0	12.0
1:20,000	5.0	10.0	15.0
1:25,000	6.25	12.5	18.75
1:50,000	12.5	25.0	37.5
1:100,000	25.0	50.0	75.0
1:250,000	62.5	125.0	187.5

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Table 11-3b. ASPRS Planimetric Feature Coordinate Accuracy Requirement (Ground X or Y in Feet) for Well-Defined Points

Target Map Scale		ASPRS Limiting RMSE in X or Y (Feet)		
1"= x ft	Ratio ft/ft	Class 1	Class 2	Class 3
5	1:60	0.05	0.10	0.15
10	1:120	0.10	0.20	0.30
20	1:240	0.2	0.4	0.6
30	1:360	0.3	0.6	0.9
40	1:480	0.4	0.8	1.2
50	1:600	0.5	1.0	1.5
60	1:720	0.6	1.2	1.8
100	1:1,200	1.0	2.0	3.0
200	1:2,400	2.0	4.0	6.0
400	1:4,800	4.0	8.0	12.0
500	1:6,000	5.0	10.0	15.0
800	1:9,600	8.0	16.0	24.0
1,000	1:12,000	10.0	20.0	30.0
1,667	1:20,000	16.7	33.3	50.0

Table 11-4a. ASPRS Topographic Elevation Accuracy Requirement for Well-Defined Points (Meters)

ASPRS Limiting RMSE in Meters

Target Contour Interval	Spot or Digital Topographic Feature Points			Terrain Model Elevation Points		
Meters	Class 1	Class 2	Class 3	Class 1	Class 2	Class 3
0.10	0.03	0.07	0.10	0.02	0.03	0.05
0.20	0.07	0.13	0.2	0.03	0.07	0.10
0.25	0.08	0.17	0.25	0.04	0.08	0.12
0.5	0.17	0.33	0.50	0.08	0.16	0.25
1	0.33	0.66	1.0	0.17	0.33	0.5
2	0.67	1.33	2.0	0.33	0.67	1.0
4	1.33	2.67	4.0	0.67	1.33	2.0
5	1.67	3.33	5.0	0.83	1.67	2.5
10	3.33	6.67	10.0	1.67	3.33	5.0

Table 11-4b. ASPRS Topographic Elevation Accuracy Requirement for Well-Defined Points (Feet)

	ASPRS Limiting RMSE in Feet					
Target Contour Interval	Topographic Feature Points			Spot or Digital Terrain Model Elevation Points		
ft	Class 1	Class 2	Class 3	Class 1	Class 2	Class 3
0.5	0.17	0.33	0.50	0.08	0.16	0.2
1	0.33	0.66	1.0	0.17	0.33	0.5
2	0.67	1.33	2.0	0.33	0.67	1.0
4	1.33	2.67	4.0	0.67	1.33	2.0
5	1.67	3.33	5.0	0.83	1.67	2.5

d. Map accuracy quality assurance testing and certification. Independent map testing is a quality assurance function that is performed independent of normal quality control during the mapping process. Specifications and/or contract provisions should indicate the requirement (or option) to perform independent map testing. Independent map testing is rarely performed for engineering and construction surveys. If performed, map testing should be completed within a fixed time period after delivery, and if performed by contract, after proper notification to the contractor. In accordance with the ASPRS standard, the horizontal and vertical accuracy of a map is checked by comparing measured coordinates or elevations from the map (at its intended target scale) with spatial values determined by a check survey of higher accuracy. The check survey should be at least twice (preferably three times) as accurate as the map feature tolerance given in the ASPRS tables, and a minimum of 20 points tested. Maps and related geospatial databases found to comply with a particular ASPRS standard should have a statement indicating that standard. The compliance statement should refer to the data of lowest accuracy depicted on the map, or, in some instances, to specific data layers or levels. The statement

should clearly indicate the target map scale at which the map or feature layer was developed. When independent testing is not performed, the compliance statement should clearly indicate that the procedural mapping specifications were designed and performed to meet a certain ASPRS map classification, but that a rigid compliance test was not performed. Published maps and geospatial databases whose errors exceed those given in a standard should indicate in their legends or metadata files that the map is not controlled and that dimensions are not to scale. This accuracy statement requirement is especially applicable to GIS databases that may be compiled from a variety of sources containing known or unknown accuracy reliability.

e. National Standard for Spatial Data Accuracy (NSSDA). The traditional small-scale "United States National Map Accuracy Standard" (Bureau of the Budget 1947) is currently (1998) being revised by the FGDC as the NSSDA. The latest draft of the NSSDA indicates it is directly based on the ASPRS standard; however, the ASPRS coordinate-based standard is converted to a 95% radial error statistic and the vertical standard is likewise converted from a one-sigma (68%) to 95% standard. The draft NSSDA defines positional accuracy of spatial data, in both digital and graphic form, as derived from sources such as aerial photographs, satellite imagery, or other maps. Its purpose is to facilitate the identification and application of spatial data by implementing a well-defined statistic (i.e., 95% confidence level) and testing methodology. As in the ASPRS standard, accuracy is assessed by comparing the positions of well defined data points with positions determined by higher accuracy methods, such as ground surveys. Unlike the above ASPRS tables, the draft NSSDA standard does not define pass-fail criteria--data and map producers must determine what accuracy exists for their data. Users of that data determine what constitutes acceptable accuracies for their applications. Unlike the ASPRS standard which uses the RMSE statistic in the X, Y, and Z planes, the NSSDA defines horizontal spatial accuracy by circular error of a data set's horizontal (X & Y) coordinates at the 95% confidence level. Vertical spatial data is defined by linear error of a data set's vertical (Z) coordinates at the 95% confidence level. ASPRS lineal horizontal accuracies in X and Y can be converted to NSSDA radial accuracy by multiplying the limiting RMSE values in Table 3 by 2.447, e.g.,

$$\text{Radial Accuracy}_{\text{NSSDA}} = 2.447 * \text{RMSE}_{\text{ASPRS-- X or Y}} \qquad \text{Eq. 11-1}$$

ASPRS 1-sigma (68%) vertical accuracies can be converted to NSSDA 95% lineal accuracy by multiplying the limiting RMSE values in Table 4 by 1.96, e.g.,

$$\text{Vertical Accuracy}_{NSSDA} = 1.96 * \text{RMSE}_{ASPRS -- z} \qquad \text{Eq. 11-2}$$

In time, it is expected that the NSSDA will be the recognized standard for specifying the accuracy of all mapping and spatial data products, and the ASPRS standard will be modified to 95% confidence level specifications.

f. Other mapping standards. When work is performed for DoD tactical elements or other Federal agencies or overseas, mapping standards other than ASPRS may be required.

11-5. Topographic and Site Plan Survey Standards. Topographic surveys and construction site plan surveys are performed for the master planning, design, and construction of installations, buildings, housing complexes, roadways, airport facilities, flood control structures, navigation locks, etc. Construction plans are performed at relatively large scales (typically ranging between 1" = 20 ft to 1" = 400 ft (1:240 to 1:4,800)) using electronic total stations, plane tables, or other terrestrial survey techniques. The ASPRS large scale mapping accuracy standards may be used for all types of ground topographic surveying methods. Guidance for performing topographic surveys is contained in a variety of commercial and government publications. Recognized industry procedural specifications, manuals, or surveying textbooks should be used for guidance in performing plane table surveys, total station surveys, and radial topographic mapping methods using kinematic GPS. For all types of topographic surveys or site plan surveys, the mapping specifications must clearly indicate the level of surface and underground feature detail to be mapped. Recommended detail scales for common types of engineering plans and topographic maps are outlined below and at the end of this chapter. Refer also to EM 1110-1-1005, Topographic Mapping.

a. Reconnaissance topographic surveys. These surveys are performed at relatively small scales--from 1" = 400 ft (1:4,800) to 1" = 1,000 ft (1:12,000). They provide a basis for general studies, site suitability decisions, or preliminary site layouts. General location of existing roads and facilities are depicted, and only limited feature and rough elevation detail is shown--5

to 10 foot contour intervals usually being adequate. Enlarged USGS 1:24,000 maps may be substituted in many cases.

b. General/Preliminary site plans. Scales from 1" = 200 ft to 1" = 400 ft (1:2,400-1:4,800). Depicts general layout arrangement of areas where construction will take place, proposed transportation systems, training areas, and existing facilities.

c. Detailed topographic surveys for construction site plan drawings. Scales from 1" = 20 ft to 1" = 200 ft--and 1 or 2-foot contour intervals. Detailed ground topographic surveys are performed to prepare base map for detailed site plans (general site layout plan, utility plan, grading plan, paving plan, airfield plan, demolition plan, etc.). Scope of mapping confined to existing/proposed building area. Used as base for subsequent as-built drawings of facilities and utility layout maps (i.e., AM/FM data bases).

d. As-Built surveys and AM/FM mapping. As-built drawings may require topographic surveys of constructed features, especially when field modifications are made to original designs. These surveys, along with original construction site plans, should be used as a base framework for an installation's AM/FM data base. Periodic topographic surveys may also be required during maintenance and repair projects in order to update the AM/FM data base.

11-6. Photogrammetric Mapping Standards and Specifications. Most smaller scale (i.e., less than 1 in = 100 ft, or 1:1,200) engineering topographic mapping and GIS data base development is accomplished by aerial mapping techniques. The ASPRS standards should be used in specifying photogrammetric mapping accuracy requirements. Procedures for developing photogrammetric mapping specifications are contained in EM 1110-1-1000, "Photogrammetric Mapping." This manual contains guidance on specifying flight altitudes, determining target scales, and photogrammetric mapping cost estimating techniques. A full contract guide specification is also contained in an appendix to the manual. More comprehensive technical guidance may be obtained from various academic publications. Currently, few aerial mapping references reflect the latest uses and potential efficiencies of GPS-controlled photogrammetry or soft copy compilation processes. In specifying photogrammetric mapping services, it is essential that feature accuracy tolerances (horizontal and vertical) be clearly identified relative to the project target mapping scale, and that

this scale be correlated with an appropriate flight altitude and the type of photogrammetric mensuration instruments used.

11-7. Construction Survey Accuracy Standards. Construction survey procedural and accuracy specifications should follow recognized industry and local practices. General procedural guidance is contained in a number of standard commercial texts. Accuracy standards for construction surveys will vary with the type of construction, and may range from a minimum of 1:2,500 up to 1:20,000. A 1:2,500 "4th Order Construction" classification is intended to cover temporary control used for alignment, grading, and measurement of various types of construction, and some local site plan topographic mapping or photo mapping control work. Lower accuracies (1:2,500-1:5,000) are acceptable for earthwork, dredging, embankment, beach fill, and levee alignment stakeout and grading, and some site plan, curb and gutter, utility building foundation, sidewalk, and small roadway stakeout. Moderate accuracies (1:5,000) are used in most pipeline, sewer, culvert, catch basin, and manhole stakeouts, and for general residential building foundation and footing construction, major highway pavement, and concrete runway stakeout work. Somewhat higher accuracies (1:10,000-1:20,000) are used for aligning longer bridge spans, tunnels, and large commercial structures. For extensive bridge or tunnel projects, 1:50,000 or even 1:100,000 relative accuracy alignment work may be required. Grade elevations are usually observed to the nearest 0.01 foot for most construction work, although 0.1 ft accuracy is sufficient for riprap placement, earthwork grading, and small-diameter pipe placement. Construction control points are typically marked by semi-permanent or temporary monuments (e.g., plastic hubs, P-K nails, iron pipes, wooden grade stakes).

11-8. Cadastral or Real Property Survey Accuracy Standards.

a. General. Many State codes, rules, statutes or general professional practices prescribe minimum technical standards for real property surveys. Corps in-house surveyors or contractors should follow applicable State technical standards for real property surveys involving the determination of the perimeters of a parcel or tract of land by establishing or reestablishing corners, monuments, and boundary lines, for the purpose of describing, locating fixed improvements, or platting or dividing parcels. Although some State standards relate primarily to accuracies of land and boundary surveys, other types of survey work may also be covered in some areas. Refer to ER 405-1-12,

Real Estate Handbook and the "Manual of Instructions for the Survey of the Public Lands of the United States" (US Bureau of Land Management 1973) for additional technical guidance on performing cadastral surveys, or surveys of private lands abutting or adjoining Government lands.

b. ALTA/ACSM standards. Real property survey accuracy standards recommended by ALTA/ACSM are contained in "Minimum Standard Detail Requirements for ALTA/ACSM Land Title Surveys," 1992. This standard was developed to provide a consistent national standard for land title surveys and may be used as a guide in specifying accuracy closure requirements for USACE real property surveys. However, it should be noted that the ALTA/ACSM standard itself not only prescribes closure accuracies for land use classifications but also addresses specific needs peculiar to land title insurance matters. The standards contain requirements for detailed information and certification pertaining to land title insurance, including information discoverable from the survey and inspection that may not be evidenced by the public records. The standard also contains a table as to optional survey responsibilities and specifications which the title insurer may require. USACE cadastral surveys not involving title insurance should follow State minimum standards; not ALTA/ACSM standards. On land acquisition surveys which may require title insurance, the decision to perform an ALTA/ACSM standard survey, including all optional survey responsibilities and specifications, should come from the project sponsor. Meeting ALTA/ACSM Urban Class accuracy standards is considered impractical for small tracts or parcels less than 1 acre in size.

11-9. Hydrographic Surveying Accuracy Standards. Hydrographic surveys are performed for a variety of engineering, construction, and dredging applications in USACE. Accuracy standards, procedural specifications, and related technical guidance are contained in EM 1110-2-1003, Hydrographic Surveying. This manual should be attached to any A-E contract containing hydrographic surveying work, and must be referenced in construction dredging contracts involving in-place measurement and payment. Standards in this manual apply to Corps river and harbor navigation project surveys, such as dredge measurement and payment surveys, channel condition surveys of inland and coastal Federal navigation projects, beach renourishment surveys, and surveys of other types of marine structures. Accuracy standards for three distinct classes of surveys are specified: (1) Contract Payment, (2) Project Condition, and (3) Reconnaissance Surveys. Standards for

nautical charting surveys or deep-water bathymetric charting surveys should conform with applicable Defense Mapping Agency (DMA), National Ocean Survey (NOS), or U.S. Naval Oceanographic Office (USNAVOCEANO) accuracy and chart symbolization criteria.

11-10. Structural Deformation Survey Standards. Deformation monitoring surveys of Corps structures require high line vector and/or positional accuracies to monitor the relative movement of monoliths, walls, embankments, etc. Deformation monitoring survey accuracy standards vary with the type of construction, structural stability, failure probability and impact, etc. Since many periodic surveys are intended to measure "long-term" (i.e., monthly or yearly changes) deformations relative to a stable network, lesser survey precisions are required than those needed for short-term structural deflection type measurements. Long-term structural movements measured from points external to the structure may be tabulated or plotted in either X-Y-Z or by single vector movement normal to a potential failure plane. Accuracy standards and procedures for structural deformation surveys are contained in EM 1110-1-1004. Horizontal and vertical deformation monitoring survey procedures are performed relative to a control network established for the structure. Ties to the National Spatial Reference System are not necessary other than for general reference, and then need only USACE Third-Order connection.

11-11. Geodetic Control Survey Standards. Geodetic control surveys are usually performed for the purpose of establishing a basic framework of the National Spatial Reference System (NSRS). These geodetic network densification survey functions are clearly distinct from the traditional engineering and construction surveying and mapping standards covered in this chapter. Geodetic control surveys of permanently monumented control points that are incorporated in the NSRS must be performed to far more rigorous standards and specifications than are control surveys used for general engineering, construction, mapping, or cadastral purposes. When a project requires NSRS densification, or such densification is a desirable by-product and is economically justified, USACE Commands should conform with published FGDC survey standards and specifications. This includes related automated data recording, submittal, project review, and adjustment requirements mandated by FGDC and the National Geodetic Survey.

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11-12. <u>Determining Surveying and Mapping Requirements for Engineering Projects</u>. General guidance for determining project-specific mapping requirements is contained in Table 11-5 at the end of this chapter. This table may be used to develop specifications for map scales, feature location tolerances, and contour intervals for typical engineering and construction projects. It is absolutely essential that surveying and mapping specifications originate from the functional requirements of the project, and that these requirements be realistic and economical. Specifying map scales or accuracies in excess of those required for project planning, design, or construction results in increased costs to USACE, local sponsors, or installations, and may delay project completion. However, the recommended standards and accuracy tolerances shown in Table 11-5 should be considered as general guidance for typical projects--variance from these norms is expected.

a. Mapping scope/limits. Mapping limits should be delineated so only areas critical to the project are covered by detailed ground or aerial surveying. The areal extent of detailed (i.e., large-scale) site plan surveys should be kept to a minimum and confined to the actual building, utility corridor, or structure area. Outside critical construction perimeters, more economical smaller scale plans should be used, along with more relaxed feature location accuracies, larger contour intervals, etc.

b. Target scale and contour interval specifications. Table 11-5 provides recommended map scales and contour intervals for a variety of engineering applications. The selected target scale for a map or construction plan should be based on the detail necessary to portray the project site. Surveying and mapping costs typically increase exponentially with larger mapping scales; therefore, specifying too large a site plan scale or too small a contour interval than needed to adequately depict the site can significantly increase project costs. Topographic elevation density or related contour intervals must be specified consistent with existing site gradients and the accuracy needed to define site layout, drainage, grading, etc., or perform quantity take offs. Photogrammetric mapping flight altitudes or ground topographic survey accuracy and density requirements are determined from the design map target scale and contour interval provided in the contract specifications.

c. Feature location tolerances. This requirement establishes the primary surveying effort necessary to delineate physical features on the ground. In most instances, a construction feature may need to be located to an accuracy well in excess of its plotted/scaled accuracy on a construction site plan; therefore, feature location tolerances should not be used to determine the required scale of a drawing or determine photogrammetric mapping requirements. In such instances, surveyed coordinates, internal CADD grid coordinates, or rigid relative dimensions are used. Table 11-5 indicates recommended positional tolerances (or precisions) of planimetric features. These feature tolerances are defined relative to adjacent points within the confines of a specific area, map sheet, or structure--not to the overall project or installation boundaries. Relative accuracies are determined between two points that must functionally maintain a given accuracy tolerance between themselves, such as adjacent property corners; adjacent utility lines; adjoining buildings, bridge piers, approaches, or abutments; overall building or structure site construction limits; runway ends; catch basins; levee baseline sections; etc. Feature tolerances should be determined from the functional requirements of the project/structure (e.g., field construction/fabrication, field stakeout or layout, alignment, locationing, etc.). Few engineering, construction, or real estate projects require that relative accuracies be rigidly maintained beyond a 1500m (5,000-ft) range, and usually only within the range of the detailed design drawing for a project/structure (or its equivalent CADD design file limit). For example, two catch basins 60 m (200 ft) apart might need to be located to 25mm (0.1 ft) relative to each other, but need only be known to ±30m (±100 ft) relative to another catch basin 10 km (6 miles) away. Likewise, relative accuracy tolerances are far less critical for small-scale GIS data elements. Actual construction alignment and grade stakeout will generally be performed to the 30mm (0.1 ft) or 3mm (0.01 ft) levels, depending on the type of construction.

d. CADD level/layer descriptors. The use of CADD or GIS equipment allows planimetric features and topographic elevations to be readily separated onto various levels or layers and depicted at any scale. Problems may arise when scales are increased beyond their originally specified values, or when so-called "rubber sheeting" is performed. It is therefore critical that these geospatial data layers, and related metadata files, contain descriptor information identifying the original source target scale and designed accuracy.

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e. Map control survey specifications. The control survey used to provide primary reference for a map or GIS product is normally performed at a higher accuracy than that required for features shown on the map. Most maps warranting an accuracy classification should be referenced to, or controlled by, conventional field surveys of Third-order accuracy. Control survey accuracies are usually measured in different units--relative distance ratios--rather than positional tolerance. Base- or area-wide mapping control procedures should be designed and specified to meet functional accuracy tolerances within the limits of the structure, building, or utility distance involved for design, construction, or real estate surveys. Higher order (i.e., First or Second-order) control surveys should not be specified for area-wide mapping or GIS definition unless a definitive functional requirement exists (e.g., military operational targeting or some low-gradient flood control projects).

f. Reference datum and coordinate system specifications. A variety of reference datums and coordinate references are used throughout Corps projects. Wide-area mapping projects should be referenced to spatial datums, plane coordinate grids, and vertical reference planes that are commonly used in the area, where practical and feasible. Small, local construction projects and cadastral surveys are usually locally referenced and need not be connected to external datums.

(1) Horizontal reference. In CONUS, maps should be horizontally referenced to either the North American Datum of 1983 (NAD 83) or 1927 (NAD 27) systems. Commands should develop plans to eventually reference all civil and military projects to the NAD 83, with connection to the GPS-based, nationwide National Spatial Reference System (NSRS) if practicable and feasible. These spatial datum coordinates should be transformed to a recognized plane coordinate system that is used in the project or installation area, such as the State Plane Coordinate System (SPCS) or Universal Transverse Mercator (UTM) grid system. The UTM grid system may be used for military operational or tactical uses in locales within and outside the continental United States (OCONUS), or on some civil projects encompassing or crossing multiple SPCS zones. Grid systems (and north arrow references) shown on maps or plans must clearly indicate the reference datum and orientation origin. When local grid systems (e.g., station-offset) are developed for detailed construction, they should be clearly distinguished from the SPCS grid.

(2) Vertical reference. In CONUS, vertical map control should be referenced to either the National Geodetic Vertical Datum, 1929 Adjustment (NGVD 29), or the North American Vertical Datum, 1988 Adjustment (NAVD 88). Independent or local survey datums and reference systems should be avoided unless required by local code, statute, or practice. Coastal navigation projects should be referenced to Mean Lower Low Water (MLLW) datum. Inland low water reference planes should be referenced to NGVD 29 or NAVD 88.

g. Plan drawing feature specifications. Mapping specifications must clearly and explicitly define the types, areal extent, and locational accuracy of features to be surveyed. The type and purpose of the map (e.g., paving plan, landscape plan, irrigation plan, etc.) should be clearly defined in the specifications in order for the field surveyor to determine which features are critical to the project. Especially critical areas, structures, or utilities should be highlighted in the technical requirements, including special accuracy tolerances for these features that may necessitate specialized ground surveying procedures. In addition, the specifications should indicate the symbology, detailing, and attributing required for recurring features, such as manholes, curbs, pavements, valves, etc. Feature symbology/delineation is specified as a function of scale--e.g., depiction of structure perimeters, centerlines, elevations, etc. For each map product, feature depiction, delineation, layering, and attributing requirements should be broken out and specified, generally following the Tri-Service discipline/drawing-based level/layer assignments or, for GIS applications, the entity set, class, attribute and domain relationships. Not all CADD levels/layers will require topographic mapping support--e.g., Interior Design. Within each level/layer, a few of the construction plans are listed below that may require field or aerial topographic mapping to delineate the features or systems on that plan. Typical features or systems within a CADD level/layer construction plan are also listed; however, this listing is not comprehensive and must be adjusted and supplemented for unique project-specific requirements, and to insure features are mapped into appropriate layers with sufficient attribute detail, as explained above.

 *

11-13. References.

ER 405-1-12
Real Estate Handbook

EM 1110-1-1000
Photogrammetric Mapping

EM 1110-1-1004
Deformation Monitoring and Control Surveying

EM 1110-1-1005
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EM 1110-2-1003
Hydrographic Surveying

ALTA/ACSM 1992
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Table 11-5. RECOMMENDED ACCURACIES AND TOLERANCES: ENGINEERING, CONSTRUCTION, AND FACILITY MANAGEMENT PROJECTS

Project or Activity	Target Map Scale SI/IP	Feature Position Tolerance Horizontal SI/IP	Feature Position Tolerance Vertical SI/IP	Contour Interval SI/IP	Survey Accuracy Hor/Vert
DESIGN, CONSTRUCTION, OPERATION & MAINTENANCE OF MILITARY FACILITIES					
Maintenance and Repair (M&R)/Renovation of Existing Installation Structures, Roadways, Utilities, Etc					
General Construction Site Plans & Specs:	1:500	100 mm	50 mm	250 mm	3rd-I
Feature & Topographic Detail Plans	40 ft/in	0.1-0.5 ft	0.1-0.3 ft	1 ft	3rd
Surface/subsurface Utility Detail Design Plans	1:500	100 mm	50 mm	N/A	3rd-I
Elec, Mech, Sewer, Storm, etc	40 ft/in	0.2-0.5 ft	0.1-0.2 ft		3rd
Field construction layout		0.1 ft	0.01-0.1 ft		
Building or Structure Design Drawings	1:500	25 mm	50 mm	250 mm	3rd-I
	40 ft/in	0.05-0.2 ft	0.1-0.3 ft	1 ft	3rd
Field construction layout		0.01 ft	0.01 ft		
Airfield Pavement Design Detail Drawings	1:500	25 mm	25 mm	250 mm	3rd-I
	40 ft/in	0.05-0.1 ft	0.05-0.1 ft	0.5-1 ft	2nd
Field construction layout		0.01 ft	0.01 ft		
Grading and Excavation Plans	1:500	250 mm	100 mm	500 mm	3rd-II
Roads, Drainage, Curb, Gutter etc.	30-100 ft/in	0.5-2 ft	0.2-1 ft	1-2 ft	3rd
Field construction layout		1 ft	0.1 ft		
Recreational Site Plans	1:1000	500 mm	100 mm	500 mm	3rd-II
Golf courses, athletic fields, etc.	100 ft/in	1-2 ft	0.2-2 ft	2-5 ft	3rd
Training Sites, Ranges, and Cantonment Area Plans	1:2500	500 mm	1000 mm	500 mm	3rd-II
	100-200 ft/in	1-5 ft	1-5 ft	2 ft	3rd
General Location Maps for Master Planning	1:5000	1000 mm	1000 mm	1000 mm	3rd-II
AM/FM and GIS Features	100-400 ft/in	2-10 ft	1-10 ft	2-10 ft	3rd
Space Management Plans	1:250	50 mm	N/A	N/A	N/A
Interior Design/Layout	10-50 ft/in	0.05-1 ft			
As-Built Maps: Military Installation Surface/Subsurface Utilities (Fuel, Gas, Electricity, Communications, Cable, Storm Water, Sanitary, Water Supply, Treatment Facilities, Meters, etc.)	1:1000 or 50-100 ft/in (Army); 1:500 or 50 ft/in (USAF)	100 mm; 0.2-1 ft	100 mm; 0.2 ft	250 mm; 1 ft	3rd-I; 3rd

Table 11-5 (Contd). RECOMMENDED ACCURACIES AND TOLERANCES: ENGINEERING, CONSTRUCTION, AND FACILITY MANAGEMENT PROJECTS

Project or Activity	Target Map Scale SI/IP	Feature Position Tolerance Horizontal SI/IP	Feature Position Tolerance Vertical SI/IP	Contour Interval SI/IP	Survey Accuracy Hor/Vert
Housing Management GIS (Family Housing, Schools, Boundaries, and Other Installation Community Services)	1:5000 100-400 ft/in	10000 mm 10-15 ft	N/A	N/A	4th 4th
Environmental Mapping and Assessment Drawings/Plans/GIS	1:5000 200-400 ft/in	10000 mm 10-50 ft	N/A	N/A	4th 4th
Emergency Services Maps/GIS Military Police, Crime/Accident Locations, Post Security Zoning, etc.	1:10000 400-2000 ft/in	25000 mm 50-100 ft	N/A	N/A	4th 4th
Cultural, Social, Historical Plans/GIS	1:5000 400 ft/in	10000 mm 20-100 ft	N/A	N/A	4th 4th
Runway Approach and Transition Zones: General Plans/Section	1:2500 100-200 ft/in	2500 mm 5-10 ft	2500 mm 2-5 ft	1000 mm 5 ft	3rd-II 3rd
Approach maps	1:5000 (H) 1:1000 (V)				
Approach detail	1:5000 (H) 1:250 (V)				
DESIGN, CONSTRUCTION, OPERATIONS AND MAINTENANCE OF CIVIL TRANSPORTATION & WATER RESOURCE PROJECTS					
Site Plans, Maps & Drawings for Design Studies, Reports, Memoranda, and Contract Plans and Specifications, Construction plans & payment					
General Planning and Feasibility Studies, Reconnaissance Reports	1:2500 100-400 ft/in	1000 mm 2-10 ft	500 mm 0.5-2 ft	1000 mm 2-10 ft	3rd-II 3rd
Flood Control and Multipurpose Project Planning, Floodplain Mapping, Water Quality Analysis, and Flood Control Studies	1:5000 400-1000 ft/in	10000 mm 20-100 ft	100 mm 0.2-2 ft	1000 mm 2-5 ft	3rd-II 3rd
Soil and Geological Classification Maps	1:5000 400 ft/in	10000 mm 20-100 ft	N/A	N/A	4th 4th
Land Cover Classification Maps	1:5000 400-1000 ft/in	10000 mm 50-200 ft	N/A	N/A	4th 4th

Table 11-5 (Contd). RECOMMENDED ACCURACIES AND TOLERANCES: ENGINEERING, CONSTRUCTION, AND FACILITY MANAGEMENT PROJECTS

Project or Activity	Target Map Scale SI/IP	Feature Position Tolerance Horizontal SI/IP	Vertical SI/IP	Contour Interval SI/IP	Survey Accuracy Hor/Vert
Archeological or Structure Site Plans & Details					
(Including Non-topographic, Close Range,	1:10	5 mm	5 mm	100 mm	2nd-I/II
Photogrammetric Mapping)	0.5-10 ft/in	0.01-0.5 ft	0.01-0.5 ft	0.1-1 ft	2nd
Cultural and Economic Resource Mapping	1:10000	10000	N/A	N/A	4th
Historic Preservation Projects	1000 ft/in	50-100 ft			4th
Land Utilization GIS Classifications	1:5000	10000 mm	N/A	N/A	4th
Regulatory Permit Locations	400-1000 ft/in	50-100 ft			4th
Socio-Economic GIS Classifications	1:10000	20000 mm	N/A	N/A	4th
	1000 ft/in	100 ft			4th
Grading & Excavation Plans	1:1000	1000 mm	100 mm	1000 mm	3rd-I
	100 ft/in	0.5-2 ft	0.2-1 ft	1-5 ft	3rd
Flood Control Structure Clearing & Grading	1:5000	2500 mm	250 mm	500 mm	3rd-II
Plans (e.g., revetments)	100-400 ft/in	2-10 ft	0.5 ft	1-2 ft	3rd
Federal Emergency Management	1:5000	1000 mm	250 mm	1000 mm	3rd-I
Agency Flood Insurance Studies	400 ft/in	20 ft	0.5 ft	4 ft	3rd
Locks, Dams, & Control Structures	1:500	25 mm	10 mm	250 mm	2nd-II
Detail Design Drawings	20-50 ft/in	0.05-1 ft	0.01-0.5 ft	0.5-1 ft	2nd/3rd
Spillways & Concrete Channels	1:1000	100 mm	100 mm	1000 mm	2nd-II
Design Plans	50-100 ft/in	0.1-2 ft	0.2-2 ft	1-5 ft	3rd
Levees and Groins: New Construction or	1:1000	500 mm	250 mm	500 mm	3rd-II
Maintenance Design Drawings	100 ft/in	1-2 ft	0.5-1 ft	1-2 ft	3rd
Construction In-Place Volume Measurement	1:1000	500 mm	250 mm	N/A	3rd-II
Granular cut/fill, dredging, etc.	40-100 ft/in	0.5-2 ft	0.5-1 ft		3rd
Beach Renourishment/Hurricane	1:1000	1000 mm	250 mm	250 mm	3rd-II
Protection Project Plans	100-200 ft/in	2 ft	0.5 ft	1 ft	3rd

Table 11-5 (Contd). RECOMMENDED ACCURACIES AND TOLERANCES: ENGINEERING, CONSTRUCTION, AND FACILITY MANAGEMENT PROJECTS

Project or Activity	Target Map Scale SI/IP	Feature Position Tolerance Horizontal SI/IP	Vertical SI/IP	Contour Interval SI/IP	Survey Accuracy Hor/Vert
Project Condition Survey Reports					
Base Mapping for Plotting Hydrographic Surveys: line maps or aerial plans	1:2500 200-1000 ft/in	10000 mm 5-50 ft	250 mm 0.5-1 ft	500 mm 1-2 ft	N/A N/A
Dredging & Marine Construction Surveys New Construction Plans	1:1000 100 ft/in	2000 mm 6 ft	250 mm 1 ft	250 mm 1 ft	N/A N/A
Maintenance Dredging Drawings	1:2500 200 ft/in	5000 mm 15 ft	500 mm 2 ft	500 mm 2 ft	N/A N/A
Hydrographic Project Condition Surveys	1:2500 200 ft/in	5000 mm 16 ft	500 mm 2 ft	500 mm 2 ft	N/A N/A
Hydrographic Reconnaissce Surveys	-	5000 m 15 ft	500 mm 2 ft	250 mm 2 ft	N/A N/A
Offshore Geotechnical Investigations Core Borings /Probings/etc.	-	5000 mm 5-15 ft	50 mm 0.1-0.5 ft	N/A	N/A 4th
Structural Deformation Monitoring Studies/Surveys					
Reinforced Concrete Structures: Locks, Dams, Gates, Intake Structures, Tunnels, Penstocks, Spillways, Bridges	Large-scale vector movement diagrams or tabulations	10 mm 0.03 ft (long-term)	2 mm 0.01 ft	N/A	N/A N/A
Earth/Rock Fill Structures: Dams, Floodwalls, Levees, etc--slope/crest stability & alignment	(same as above)	30 mm 0.1 ft (long term)	15 mm 0.05 ft	N/A	N/A N/A
Crack/Joint & Deflection Measurements: piers/monoliths--precision micrometer	tabulations	0.2 mm 0.01 inch	N/A	N/A	N/A N/A

Table 11-5 (Contd). RECOMMENDED ACCURACIES AND TOLERANCES: ENGINEERING, CONSTRUCTION, AND FACILITY MANAGEMENT PROJECTS

Project or Activity	Target Map Scale SI/IP	Feature Position Tolerance Horizontal SI/IP	Feature Position Tolerance Vertical SI/IP	Contour Interval SI/IP	Survey Accuracy Hor/Vert
REAL ESTATE ACTIVITIES: ACQUISITION, DISPOSAL, MANAGEMENT, AUDIT					
Maps, Plans, & Drawings Associated with Military and Civil Projects					
Tract Maps, Individual, Detailing					
Installation or Reservation Boundaries,	1:1000	10 mm	100 mm	1000 mm	3rd-I/II
Lots, Parcels, Adjoining Parcels, and	1:1200 (Army)				3rd
Record Plats, Utilities, etc.	50-400 ft/in	0.05-2 ft	0.1-2 ft	1-5 ft	
Condemnation Exhibit Maps	1:1000	10 mm	100 mm	1000 mm	3rd-I/II
	50-400 ft/in	0.05-2 ft	0.1-2 ft	1-5 ft	3rd
Guide Taking Lines/Boundary Encroachment	1:500	50 mm	50 mm	250 mm	3rd-I/II
Maps: Fee and Easement Acquisition	20-100 ft/in	0.1-1 ft	0.1-1 ft	1 ft	3rd
General Location or Planning Maps	1:24000	10000 mm	5000 mm	2000 mm	N/A
	2000 ft/in	50-100 ft	5-10 ft	5-10 ft	4th
GIS or Land Information System (LIS) Mapping, General					
Land Utilization and Management, Forestry	1:5000	10000 mm	N/A	N/A	3rd
Management, Mineral Acquisition	200-1000 ft/in	50-100 ft			3rd
Easement Areas and Easement	1:1000	50 mm	50 mm	N/A	3rd
Delineation Lines	100 ft/in	0.1-0.5 ft	0.1-0.5 ft		3rd
HAZARDOUS, TOXIC, RADIOACTIVE WASTE (HTRW) SITE INVESTIGATION, MODELING, AND CLEANUP					
General Detailed Site Plans	1:500	100 mm	50 mm	100 mm	2nd-I/II
HTRW Sites, Asbestos, etc.	5-50 ft/in	0.2-1 ft	0.1-0.5 ft	0.5-1 ft	2nd/3rd
Subsurface Geotoxic Data Mapping	1:500	100 mm	500 mm	500 mm	3-II
and Modeling	20-100 ft/in	1-5 ft	1-2 ft	1-2 ft	3rd
Contaminated Ground Water	1:500	1000 mm	500 mm	500 mm	3rd-II
Plume Mapping/Modeling	20-100 ft/in	2-10 ft	1-5 ft	1-2 ft	3rd
General HTRW Site Plans &	1:2500	5000 mm	1000 mm	1000 mm	3rd-II
Reconnaissance Mapping	50-400 ft/in	2-20 ft	2-20 ft	2-5 ft	3rd

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EXPLANATORY NOTES FOR COLUMNS IN TABLE 11-5:

1. Target map scale is that contained in CADD, GIS, and/or AM/FM layer, and/or to which ground topo or aerial photography accuracy specifications are developed. This scale may not always be compatible with the feature location/elevation tolerances required. In many instances, design or real property features are located to a far greater relative accuracy than that which can be scaled at the target (plot) scale, such as property corners, utility alignments, first-floor or invert elevations, etc. Coordinates/elevations for such items are usually directly input into a CADD or AM/FM data base.

2. The feature position or elevation tolerance of a planimetric feature is defined at the 95% confidence level. The positional accuracy is relative to two adjacent points within the confines of a structure or map sheet, not to the overall project or installation boundaries. Relative accuracies are determined between two points that must functionally maintain a given accuracy tolerance between themselves, such as adjacent property corners; adjacent utility lines; adjoining buildings, bridge piers, approaches, or abutments; overall building or structure site construction limits; runway ends; catch basins; levee baseline sections; etc. The tolerances between the two points are determined from the end functional requirements of the project/structure (e.g., field construction/fabrication, field stakeout or layout, alignment, locationing, etc.).

3. Horizontal and vertical control survey accuracy refers to the procedural and closure specifications needed to obtain/maintain the relative accuracy tolerances needed between two functionally adjacent points on the map or structure, for design, stakeout, or construction. Usually 1:10,000 Third-Order (I) control procedures (horizontal and vertical) will provide sufficient accuracy for most engineering work, and in many instances of small-scale mapping or GIS rasters, Third-Order, Class II methods and Fourth-Order topo/construction control methods may be used. Base- or area-wide mapping control procedures shall be specified to meet functional accuracy tolerances within the limits of the structure, building, or utility distance involved for design or construction surveys. Higher order control surveys shall not be specified for area-wide mapping or GIS definition unless a definitive functional requirement exists (e.g., military operational targeting or some low-gradient flood control projects).

Appendix A
References

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Appendix B
Abbreviations and Acronyms List

AA	Analysis of Alternatives
AAG	Association of American Geographers
ACSM	American Congress on Surveying and Mapping
ADPE	Automatic Data Processing Equipment
AM/FM	Automated Mapping/Facilities Management
ANSI	American National Standards Institute
APR	Agency Procurement Request
ARPA	Advanced Research Projects Agency
ASPRS	American Society for Photogrammetry and Remote Sensing
CAD	Computer Aided Design
CAD2	Computer Aided Design 2 Contract
CADD	Computer-aided Design and Drafting
CD-ROM	Compact Disk-Read Only Memory
CEAP	Corps of Engineers Automation Plan
CLIN	Contract Line Item Number
CPU	Central Processing Unit
CRREL	Cold Regions Research and Engineering Laboratory
DAT	Digital Audio Tape
DLG	Digital Line Graph
DMA	Defense Mapping Agency
DPA	Delegation of Procurement Authority
EO	Executive Order
EAB	Environmental Advisory Panel
EC	Engineer Circular
EPA	Environmental Protection Agency
ESRI	Environmental Systems Research Institute
FEMA	Federal Emergency Management Agency
FGDC	Federal Geographic Data Committee
FIPS	Federal Information Processing Standards
FIPSPUB	Federal Information Processing Standards Publication
FIRMR	Federal Information Resources Management Regulation
FTP	File Transport Protocol
FY	Fiscal Year
GAO	Government Accounting Office
GB	gigabytes (1024 MB)
GD/S	Geospatial Data/Systems
GDS	Geospatial Data System
GIS	Geographic Information System
GOSIP	Government Open Systems Interconnection Profile (FIPS 146-1)
GRASS	Geographic Resources Analysis Support System
HQUSACE	Headquarters United States Army Corps of Engineers
HTRW	Hazardous, Toxic, and Radioactive Waste
IAW	In Accordance With
IBM	International Business Machines
IEEE	Institute of Electrical and Electronic Engineers

IP	Implementation Plan
ISO	International Standards Organization
ITU	International Telecommunications Union
KB	kilobytes (1024 bytes)
LAN	Local Area Network
LCMIS	Life Cycle Management of Information Systems
LIS	Land Information System
LMV	Lower Mississippi Valley Division
MB	megabytes (1024 KB)
MIL-STD	Military Standard
MMDF	Multichannel Memorandum Distribution Facility
MSDOS	Microsoft Disk Operating System
NASA	National Aeronautics and Space Administration
NAVFAC	Naval Facilities
NFS	Network File System
NIST	National Institute of Standards and Technology
NSDC	National Spatial Data Clearinghouse
NSDI	Natinal Spatial Data Infrastructure
NSGIC	National States Geographic Information Council
O&M	Operation and Maintenance
OGF	Open GIS Foundation
OGIS	Open Geodata Interoperability Standard/Specification
OMB	Office of Management and Budget
OS	Operating System

OSI	Open Systems Interconnection
PC	Personal Computer (IBM PC-compatible)
POC	Point of Contact
PROSPECT	Proponent Sponsored Engineer Corps Training
QA	Quality Assurance
RA	Requirements Analysis
RAM	Random Access Memory
SDTS	Spatial Data Transfer Standard
SMC	Small Multi-user Computer
SRS	Systems Requirements Specification
TAC	Transatlantic Center
TBD	To Be Decided
TCP/IP	Transmission Control Protocol/Internet Protocol
TEC	Topographic Engineering Center
URISA	Urban and Regional Information Systems Association
URL	Universal Resource Locator
USACE	United States Army Corps of Engineers
USGS	United States Geological Survey
VPF	Vector Product Format
WAIS	Wide Area Information Servers
WAN	Wide Area Network
WES	Waterways Experiment Station
WRSC	Water Resources Support Center
WWW	World WideWeb

Appendix C
Internet Information Sites

C-1. USACE Resources Available on the Internet

The primary resource for information regarding GD&S issues in the Corps of Engineers is the USACE Clearinghouse Node. The Internet URL for this site is:

U.S. Army Corps of Engineers National Geospatial Data Clearinghouse Node.

http://corps_geo1.usace.army.mil

Additional Internet sites with GD&S information within the Corps are:

Tri-Services CADD/GIS Technology Center.

http://mr2.wes.army.mil

Headquarters, U.S. Army Corps of Engineers.

http://www.usace.army.mil

U.S. Army Corps of Engineers Cold Regions Research and Engineering Laboratory

http://www.usace.army.mil/crrel

U.S. Army Corps of Engineers Waterways Experiment Station

http://www.wes.army.mil/Welcome.html

U.S. Army Corps of Engineers Construction Engineering Research Laboratory

http://www.cecer.army.mil:80/welcome.html

U.S. Army Corps of Engineers Topographic Engineering Center

http://www.tec.army.mil

GRASS GIS home page

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GRASS GIS source code

ftp://moon.cecer.army.mil/grass

C-2. GD&S Information Sites on the Internet

An up-to-date listing of Internet resources for GD&S materials including databases, software, source code, etc. is maintained by Queens University in Canada. This list is complied from user submissions and other net resources. It has not been verified, so do not be surprised if one of the sites refuses access or does not exist. The list is periodically updated and is available at:

ftp://gis.queensu.ca/pub/gis/docs/gissites.txt and as a WWW document at:

ftp://gis.queensu.ca/pub/gis/docs/gissites.html

The 9 December 1995 list is provided below:

Last Update: December 9, 1995.

Compiled from user submissions and various net resources.

E-mail additions to Michael McDermott:

mcdermom@gisdog.gis.queensu.ca

AGU - American Geophysical Union home page.

http://earth.agu.org/kosmos/homepage.html

AIDS Epidemic animations of the U.S., 1981-1993 - CIESEN.

http://www.ciesin.org/datasets/cdc-nci/cdc-nci.html

Analytical Spectral Devices - makers of field portable spectroradiometers.

http://www.asdi.com/asd/

Arc/Info AML FTP site.

ftp://wigeo.wu-wien.ac.at/pub/acdgis-l/aml

Arc/Info to IDRISI AML conversion routines.

ftp://mars.uoregon.edu/pub

Arc/Info tutorial (requires UNIX).

http://boris.qub.ac.uk/shane/arc/archome.html

ARGUS Census Map USA Page.

http://www.tcel.com/~argus

Arizona Geographic Information Council.

http://www.state.az.us/gis3/agic/agichome.html

Atlantis Scientific home page, includes info on ERGOvista (RS program).

http://www.atlsci.com

Atlas GIS mailing list and FAQ home page.

http://www.csn.net:80/~rdandrea

Australian National University - Bioinformatics.

http://life.anu.edu.au:80/

Australian National University - Landscape Ecology Services.

http://life.anu.edu.au/landscape_ecology/landscape.html

BADGER, NASA's Bay Area Digital Geo-Resource Project home page.

http://www.svi.org/badger.html

Bibliographic search page on the environment from the USGS.

http://waisqvarsa.er.usgs.gov/public/biblio.html

Blackwell Publishing's catalogue of Geography books.

http://www.cityscape.co.uk/bookshop/bwcat.html

BLM (US) - Various conversion programs and datasets.

ftp://ftp.blm.gov/pub/gis/index.html

Boulder County, Colorado GIS landuse department.

http://www.boco.co.gov/gislu

British Ordnance Survey home page.

http://www.open.gov.uk/ordsurv/oshome.html

California - Teale Data Center GIS data library (Arc/Info format).

http://gis.gislab.teale.ca.gov

CALMIT - Center for Advanced Land Management Information Technologies, University of Nebraska

http://ianrwww.unl.edu/ianr/calmit/calmit.htm

CalTech (US) - Earth Viewer.

http://www.ugcs.caltech.edu/htbin/earth

Canada Centre for Remote Sensing.

http://www.ccrs.nrcan.gc.ca

Canadian WWW servers top level list and virtual map.

http://www.sal.ists.ca/services/w3_can/www_index.html

CANSPACE - Canadian Space Geodesy Forum archive (lots of GPS info).

gopher://unbmvs1.csd.unb.ca:1570/1EXEC%3aCANSPACE

CARIS GIS/Universal Systems Ltd. home page.

http://www.universal.ca

Cartography Resources on the Web - maintained by Jeremy Crampton.

http://geog.gmu.edu/gess/jwc/cartogrefs.html

CAST - The Center for Advanced Spatial Technologies, University of Arkansas

http://www.cast.uark.edu/

Catalogue of Digital Elevation Data (maintained by Bruce Gittings).

http://www.geo.ed.ac.uk/home/ded.html

CCM - Canada Centre for Mapping.

http://ccm-10.ccm.emr.ca/

CCM - Selected bibliography including hypermedia and geographic data.

http://ccm-10.ccm.emr.ca/publications/ccmpubs.html

CDIAC - Carbon Dioxide Information Analysis Center. Contains climate datasets.

ftp://suns01.esd.ornl.gov/pub

Census Data archive (US) at the Lawrence Berkley Laboratory.

ftp://cedrcd.lbl.gov/data1/merrill/docs/LBL_census.html

Center for Remote Sensing and Spatial Analysis - Rutgers University.

http://deathstar.rutgers.edu/

Centre for Earth Observation (CEO) of the European Community home page.

http://www.ceo.org

Centre for Earth Observation (CEO) European-Wide Service Exchange (EWSE).

http://ewse.ceo.org

CERL/GRASS welcome file.

http://www.cecer.army.mil:80/welcome.html

CERN - WWW Virtual Library on the subject of Geography.

http://info.cern.ch/hypertext/DataSources/bySubject / Geography/Overview.html

Chartwrite AB's Data-on-the-Map - an easy to use mapping/GIS program.

http://www.chartwrite.se

CIESIN - The Consortium for International Earth Science Information Network.

http://infoserver.ciesin.org

CIA World Databank.

ftp://gatekeeper.dec.com/pub/graphics/data/cia-wdb

CIA World Fact Book.

http://www.ncsa.uiuc.edu:8001/cmns-moon.think.com:201/CIA?

CIA World Map database and lat/long coord. of major U.S. cities.

ftp://ftp.cs.toronto.edu/doc/geography

comp.infosystems.gis Usenet newsgroup.

news://comp.infosystems.gis

CMU Maps home page (US).

http://www.cs.cmu.edu/afs/cs.cmu.edu/user/maps / www/home.html

DCW - Digital Chart of the World (US data).

http://waisqvarsa.er.usgs.gov/public/dcwindex.html

DCW - Digital Chart of the World - documentation.

http://waisqvarsa.er.usgs.gov/public/dcw/dcwinfo.html

Delorme Mapping Corporation.

http://www.delorme.com

Deutsches Klimarechenzentrum (German Climate Computer Centre).

http://www.dkrz.de

Digital Map Library, Queen's University (restricted to Queens users only).

ftp://ftp.ccs.queensu.ca/pub/gismaps

Digitized Outline of the U.S./world and 1980 Census Country Boundaries.

ftp://ftp.uu.net/graphics/maps

DLG, DEM, GBF-DIME, DTM and TIGER files (US).

ftp://spectrum.xerox.com/pub/map

DLG-E Information.

ftp://sdts.er.usgs.gov/pub/dlge

Earth Observatorium - a CD-ROM on NASA's "Mission to Planet Earth" (STS-59).

http://www.csn.net/malls/rmdp/ee.html

Earth Observations Lab, Institute for Space and Terrestrial Science (CAN).

http://eol.ists.ca/Welcome.html

Earthquake info from the USGS in Menlo Park, CA

http://quake.wr.usgs.gov/

EINet Galaxy's Pointers to Geography Resources.

http://galaxy.einet.net/galaxy/Science/Geography.html

Environmental Info Resources Top Level List (maintained by David Crossley).

http://kaos.erin.gov.au/other_servers/other_servers.html

EPA - Environmental Protection Agency home page (US).

http://www.epa.gov/

EPA - National GIS Program.

http://www.epa.gov/docs/ngispr/natgispr.html

EPA - National library database (US).

telnet://epaibm.rtpnc.epa.gov

EPA - Office of Policy, Planning and Evaluation, GIS resources page.

http://www.epa.gov/docs/oppe/spatial.html

EPA - Region 10: AK, ID, OR, and WA - includes map and data library.

http://www.epa.gov/region10/www/r10.html

EPI-MAP-US Center for Disease Control GIS source and data.

ftp://oak.oakland.edu/pub/msdos/mapping

ERIN - Australian Environmental Resources Information Network.

http://kaos.erin.gov.au/erin.html

EROS - US Earth Resources Observation Systems Remote Sensing Data Center.

http://sun1.cr.usgs.gov/eros-home.html

ETOPO5 Sample colour shaded relief maps of the U.S.

ftp://src.doc.ic.ac.uk/geology/maps/gifmaps

ESIS - European Space Information System.

http://mesis.esrin.esa.it/html/esis.html

Faculty of Regional Science and Architecture, University of Vienna.

http://info.archlab.tuwien.ac.at

Fed. Institute of Technology GERMINAL Project - GIS/ Environment Management and Planning.

http://dgrwww.epfl.ch/GERMINAL/Germinal.html

FGDC (US) - Spatial Metadata Standard text source.

ftp://waisqvarsa.er.usgs.gov/wais/docs

FGDC - US Manual of Federal Geographic Data Products.

http://info.er.usgs.gov/fgdc-catalog/title.html

Finland National Land Survey Geographic Information Centre.

http://www.mmh.fi/

Fish and Wildlife Service (US) National Wetlands Inventory FTP site.

ftp://192.189.43.33/

Forestland data for the US (Arc/Info format).

http://www.epa.gov/docs/grd/forest_inventory/

Gazetteer for the U.S.

http://wings.buffalo.edu/geogw

GCMD - Global Change Master Directory.

telnet://GCNET@132.156.47.218

GCTP - USGS General Cartographic Transformation Package.

ftp://isdres.er.usgs.gov/.USGS.GCTP

GDE Systems Inc. - Photogrammetric, cartographic and image processing systems.

http://www.gdesystems.com

GeoData Institute at the University of Southampton.

http://www.geodata.soton.ac.uk/

Geodetic Survey of Canada.

http://www.geod.emr.ca/

GeoEAS - EPA (US) geostatistical library source.

ftp://math.arizona.edu/pub/geostat

Geographic Designs Inc. - Software, research and consulting services.

http://www.geodesigns.com

Geographic name server for USA (enter name to search).

telnet://martini.eecs.umich.edu:3000

Geography and GIS Servers Listing - maintained at Utrecht University.

http://www.frw.ruu.nl/nicegeo.html

Geologic fault database for the U.S.

ftp://alum.wr.usgs.gov/pub/map

Geologic Map of U.S. Pacific Northwest Continental Margin.

http://mustang.oce.orst.edu

Geological Survey of Canada On-Line Library.

telnet://opac@geoinfo.gsc.emr.ca

Geological Survey Bureau of the Iowa Department of Natural Resources GIS page.

http://samuel.igsb.uiowa.edu

Geological Survey of Japan home page.

http://www.aist.go.jp:7128/

Geomatics International (Canada) home page.

http://www.geomatics.com/

GEOname Digital Gazetteer - CD with four million non-US placenames.

http://www.gdesystems.com/IIS/SlipSheets/GEONAME.html

GeoVu - NGDC MS-Windows GIS/RS data visualization program.

ftp://ftp.ngdc.noaa.gov/Acess_Tools

GIAL - Geographic Information and Analysis Laboratory: NCGIA Buffalo.

http://zia.geog.buffalo.edu/GIAL/netgeog.html

GIF Images of Israel and Surrounding Countries.

gopher://israel-info.gov.il:70/11/gifs

GIS Job Clearinghouse.

http://walleye.forestry.umn.edu:70/0/www/main.html

GIS Job Announcement Gopher Server.

gopher://nero.aa.msen.com:70

GIS Master Bibliography Project (maintained by Duane Marble).

http://thoth.sbs.ohio-state.edu/

GIS Resource Pointers on the Internet (maintained by Matthias Werner).

http://www.laum.uni-hannover.de/gis/gisnet/gisnet.html

GIS Software Listing (maintained by Oliver Weatherbe).

http://triton.cms.udel.edu/~oliver/gislist.html

GIS User's Guide to Internet Tools (maintained by S. Murnion and G. Munroe).

file://jupiter.qub.ac.uk/pub/GIS/GIS.html

GIS-L and comp.infosystems.gis FAQ listing (maintained by Lisa Nyman).

ftp://abraxas.adelphi.edu/pub/gis/FAQ

GIS-L and comp.infosystems.gis FAQ listing alternate site.

ftp://ftp.census.gov/pub/geo/gis-faq.txt

GIS/Remote Sensing/GPS/Geosciences top level page at Texas A&M (very good).

http://ageninfo.tamu.edu/geoscience.html

GLIS - EROS Data Center Global Land Information System (X Windows only).

telnet://xglis.cr.usgs.gov:5060/

GLIS - Global Land Information System.

http://edcwww.cr.usgs.gov/glis/glis.html

GMT - Generic Mapping Tools software source.

ftp://kiawe.soest.hawaii.edu/pub/gmt

GPS Information Center, U.S. Coast Guard (login: select database 34).

telnet://fedworld.gov

GPS Information Sources (maintained by Richard Langley).

ftp://unbmvs1.csd.unb.ca/PUB.CANSPACE.GPS . INFO.SOURCES

GPS satellite current constellation status.

finger gps@geomac.se.unb.ca

GRASS GIS home page.

http://www.cecer.army.mil/grass/GRASS.main.html

GRASS GIS source code for UNIX platforms including LINUX.

ftp://moon.cecer.army.mil/grass

GSLIB - Stanford U. Geostatistical Library source (FORTRAN).

ftp://wiener.stanford.edu/gslib1.31

HOLIT - Israel Ecological & Environmental Information System.

http://vms.huji.ac.il/www_teva/db000.html

IDMSP - Raster image display/manipulation software.

ftp://hypatia.gsfc.nasa.gov/pub/software/imdisp

IDRISI-L FTP site (donated data, modules and more).

ftp://midget.towson.edu/idrisi

Illinois Natural History Survey GIS.

http://www.inhs.uiuc.edu:70/0h/igis/igismain

ImageNet - Landsat, SPOT and Russain RS Imagery archive/browser.

http://www.coresw.com/

Infotech Enterprises - GIS data conversion and programming home page.

http://infotech.stph.net

INGIS - The Indiana Geographic Information System Server.

http://ingis.acn.purdue.edu/

Institute for Photogrammetry and Remote Sensing, University of Karlsruhe (GER).

http://www.laum.uni-hannover.de/uis/uis.html

Institute for Photogrammetry, University. of Stuttgart (GER).

http://hpux.bauingenieure.uni-stuttgart.de

Institute for Regional Planing and Regional Science, University of Hannover (GER).

http://www.laum.uni-hannover.de/gis/gisnet / wwwgis.html

Institute for Space and Terrestrial Science (CAN).

http://www.ists.ca/Welcome.html

Institute of Geography, University of Salzburg (restricted access).

http://www.edvz.sbg.ac.at/geo/home.html

Intergraph Corporation.

http://www.intergraph.com

IOWA Department of Natural Resources FTP site (US).

ftp://gsb.igsb.uiowa.edu/

ISL - Information Systems Workgroup search page (several search engines).

http://www_is.cs.utwente.nl:8080/cgi-bin/local / nph-susi1.pl

Japan Virtual Map.

http://www.ntt.jp/japan/map

JHU/APL Digital relief maps of U.S image browser.

http://ageninfo.tamu.edu/apl-us/

JHU/APL ETOPO5 Digital relief maps of U.S.

http://www.doc.ic.ac.uk/public/geology/maps/gifmaps/

KHOROS - visualizaion software development environment source (working?).

ftp://pprg.eece.unm.edu/pub/khoros/release

Laboratory for Telecommunication and Remote Sensing Technology - TUDelft, NL.

http://tudedv.et.tudelft.nl/www/ttt/rs/rs_home.html

Lat/Long file of major world cities (coarse).

ftp://gis.queensu.ca/pub/gis/city/geocity.zip

Library of Congress (US) country information.

http://lcweb.loc.gov/homepage/country.html

Linnet Geomatics International (Canada) - includes LIS/GIS data repository.

http://www.linnet.ca/linnet/

macGIS gopher server.

gopher://dslmac.uoregon.edu:70

MapInfo FAQ (maintained by John McCombs).

ftp://ftp.csn.org/mapinfo/mi_faq.zip

MapInfo “maplication” programs.

ftp://ftp.csn.org/mapinfo

McGill University Department of Earth Sciences (CAN).

http://stoner.eps.mcgill.ca

Meta-Index of GIS related material on the Web (very comprehensive).

http://abacus.bates.edu/~nsmith/General / Resources-GIS.html

Meteorological, Geophysical and Oceanographic top level data list.

http://www.cis.ohio-state.edu/hypertext/faq/usenet / weather/top.html

MIT - Earth Resources Laboratory.

http://www-erl.mit.edu

Montana Natural Resource GIS data library (Arc/Info format).

http://nris.msl.mt.gov/gis/mtmaps.html

MOSS - PC GIS source.

ftp://ftp.csn.org/COGS/MOSS

NAISMap - National Atlas Information Service (Canada) WWW-based GIS (very impressive).

http://ellesmere.ccm.emr.ca/naismap/naismap.html

NASA - home page.

http://hypatia.gsfc.nasa.gov/NASA_homepage.html

NASA - Images from the Voyager and Magellan missions.

ftp://explorer.arc.nasa.gov/pub/SPACE

NASA - NASA Internet connections top level list.

http://krakatoa.jsc.nasa.gov/NASAInternetConnection.html

Naval Observatory Automated Data Service.

telnet://ads@tycho.usno.navy.mil

NCAR - National Center for Atomospheric Research data support archive.

ftp://ncardata.ucar.edu/

NCAR - US National Center for Atmospheric Research home page.

http://www.ucar.edu

NCEER - Seismic Activity Server.

telnet://strongmo:nceer@duke.ldgo.columbia.edu:23/8

NCGIA FTP server at U. of Maine.

ftp://grouse.umesve.maine.edu/pub/NCGIA/

NCGIA WWW server as U. of Maine.

http://blackbird.umesve.maine.edu/home_page.html

NCGIA FTP server at U.C. Santa Barbara (UCSB).

ftp://ncgia.ucsb.edu/pub

NCGIA WWW server at U.C. Santa Barbara (UCSB).

http://www.ncgia.ucsb.edu/

NCGIA WWW server SUNY Buffalo.

http://zia.geog.buffalo.edu/

NCSU - NCSU library gopher: GIS.

gopher://dewey.lib.ncsu.edu:70/11/library/disciplines/geogrphy/gis

NGDC - National Geophysical Data Center (US) home page.

http://www.ngdc.noaa.gov/ngdc.html

Norwegian Institute of Technology GIS pointer page.

http://www.iko.unit.no/gis/gisen.html

NRC - Natural Resources Canada (Energy, Mines and Resources Canada).

http://www.emr.ca/

NRC - Natural Resources Canada On-line Library.

telnet://opac@canlib.emr.ca

NSCA - WWW search engine meta-index (very useful).

http://www.ncsa.uiuc.edu/SDG/Software/Mosaic/Demo/metaindex.html

NSSDC On-line Data and Information Server (including NASA master dir).

telnet://nodis@nssdca.gsfc.nasa.gov

Ocean Mapping Group, U. of New Brunswick (includes sea-floor fly-over MPEGs).

http://www.omg.unb.ca/omg/

Online Career Center (useful for both GIS job searchers and GIS employers).

gopher://gopher.msen.com

On-line Resources for Earth Scientists FAQ (maintained by Bill Thoen).

ftp://ftp.csn.net/COGS/ores.txt

Oregon's State Service Center for Geographic Information Systems.

http://www.sscgis.state.or.us

ORNL - Oak Ridge Nat. Laboratory (US) fire simulation program EMBYR.

http://jupiter.esd.ornl.gov/ern/embyr/embyr.html

OSU - Ohio State University (US).

http://www.cis.ohio-state.edu

OZGis - OZ GIS source (msdos and windows).

ftp://oak.oakland.edu/pub/msdos/mapping

Pacific GIS - A non-profit organization devoted to public access GIS.

http://www.ptialaska.net/~pacgisak

Paleomagnetic database for the world. Includes DTM images.

ftp://earth.eps.pitt.edu/pub

PARC - Xerox Palo Alto Research Center map viewer.

http://pubweb.parc.xerox.com/map

PCI home page.

http://www.pci.on.ca/

PHARUS - Phased Array Universal SAR project, TUDelft, FEL, & NLR, Netherlands.

http://tudedv.et.tudelft.nl/www/ttt/rs/pharus / pharus_home.html

Planet Earth Home Page (a very good top level WWW server).

http://white.nosc.mil/info.html

PROJ - projection conversion program source.

ftp://charon.er.usgs.gov/pub/PROJ.4

Project GeoSim - Geography education software.

http://geosim.cs.vt.edu/index.html

REGIS - UC Berkeley Research Program in Environmental Planning and GIS.

http://www.ced.berkeley.edu

Remote Sensing Catalogue of the WWW Virtual Library (maint. by VTT-RS Group).

http://www.vtt.fi/aut/ava/rs/virtual/

Resources for Cartorgraphy, GIS and Remote Sensing (maint. by Michael Ritter).

http://www.uwsp.edu/acaddept/geog/cart.htm

Rice University (US) - Geography, software, GIS info and more.

gopher://riceinfo.rice.edu/11/Subject/Geography

RSD - NASA's Public Use of Remote Sensing Data home page.

http://camille.gsfc.nasa.gov/rsd/

Ryerson University GIS and Geography home page.

http://www.acs.ryerson.ca/~gta

SAIF - Canadian Spatial Archive Interchange Format specifications.

http://www.wimsey.com/~infosafe/saif/saifHome.html

Satellite images of Japan and the Pacific.

http://www.ntt.jp/japan/GMS

Satellite images, various.

http://web.nexor.co.uk/places/satelite.html

SDTS - US Spatial Data Transfer Standard (FIPS 173).

ftp://sdts.er.usgs.gov/pub/sdts/

Sea-surface temperature data (near real-time).

ftp://satftp.soest.hawaii.edu/pub/avhrr/images

SOEST - Generic Mapping Tool (GMT).

http://www.soest.hawaii.edu/soest/about.ftp.html

Software Support Laboratory: Space and Earth Scientists Software Tools.

http://sslab.colorado.edu:2222/ssl_homepage.html

Space and Astronomy related FAQs.

http://www.cis.ohio-state.edu/hypertext/faq/usenet / spae/top.html

SRSC - Space Remote Sensing Center, Inst. for Technology Development (US).

http://www.itd.com

STIS - US NSF Science and Technology Information Service.

telnet://public@stis.nsf.gov

Strategic Mapping Inc. home page.

http://www.stratmap.com

Synmap Information Technologies, NS, Canada - inc. geogdata for South America.

http://www.isisnet.com/synmap/index.html

TAMU - Sample ETOP5 dataset relief images (working?).

ftp://ageninfo.tamu.edu/11/apl-us

TAMU - Texas A&M U. version of LINUX (access?).

ftp://sc.tamu.edu

Thomas Bros. Maps home page.

http://www.thomas.com/

TIGER - US Census TIGER files mapping service.

http://tiger.census.gov

Texas GIS Planning Council home page (inc. Texas GIS data).

file://sun.stac.dir.texas.gov/DIR/gis.html

UNC - U of North Carolina Inst. for Transportation Research and Education.

http://itre.uncecs.edu

UNEP GRID database listing (GRIDLIST.DOS, GRIDLIST.WP5).

ftp://camelot.es.anl.gov/pub

UNIDATA gopher server with weather summary maps for the U.S. and world.

gopher://groucho.unidata.ucar.edu:70/11/Images

University of Washington Geophysics program home page.

http://www.geophys.washington.edu/

US Bureau of the Census home page.

http://www.census.gov/

US Fish and Wildlife service - National Wetlands Inventory FTP site.

ftp://192.189.43.33/

US Fish and Wildlife service - National Wildlife Refuge System WWW server.

http://bluegoose.arw.r9.fws.gov/

US GeoData index files.

ftp://greenwood.cr.usgs.gov/pub/esic

USGS - United States Geological Survey home page.

http://info.er.usgs.gov

USGS - United States Geological Survey WAIS.

wais://info.er.usgs.gov/USGS-Server

USGS beginners tutorial on GIS.

http://info.er.usgs.gov/research/gis/title.html

USGS Distributed Spatial Data Library.

file://waisqvarsa.er.usgs.gov/wais/home.html

USGS federal government data products catalog.

http://info.er.usgs.gov/fgdc-catalog/title.html

USGS geologic mapping and volcanic hazards assessment home page.

http://vulcan.wr.usgs.gov/Projects/Mapping+hazards/framework.html

USGS GIS home page.

http://info.er.usgs.gov/research/gis/title.html

USGS GISLab FTP site - 1:250000 DEM depot.

ftp://edcftp.cr.usgs.gov/pub/data/DEM/250

USGS Global Land Information System (GLIS) - LANDSAT Archive.

telnet://glis.cr.usgs.gov:guest

USGS - National Geospatial Data Clearinghouse.

http://nsdi.usgs.gov/nsdi/index.html

USGS/EROS FTP site - DEM and DLG file depot.

ftp://edcftp.cr.usgs.gov/pub/data

USGS - Technical Standards and Cartographic Software.

ftp://nmdpow9.er.usgs.gov/public

US National Biological Service, Environmental Management Technical Center.

http://www.emtc.nbs.gov/

US National Parks Service - digital data archive (Arc/Info format).

ftp://ftp.its.nps.gov/pub/park_boundaries

U.S. Weather Service Hydrologic RS Center - AVHRR sample images and viewers.

ftp://snow.nohrsc.nws.gov/AVHRR_sample

Virtual Tourist Map of WWW information services around the world.

http://wings.buffalo.edu/world

VPFVIEW (DCW) program for UNIX (Sun workstations) - DND Canada.

ftp://jupiter.drev.dnd.ca/pub/gis/vpfview

VTT Automation, Space Technology and Remote Sensing Services.

http://www.vtt.fi/aut/ava/rs/

Weather information, Japan.

gopher://gan.ncc.go.jp:70/11/INFO/weather

Weather map - U.S.

gopher://meteor.atms.purdue.edu:70/1/satellite

Weather satellite and surface analysis images for North America.

ftp://wuarchive.wustl.edu/multimedia/images/wx

WOCE - World Ocean Circulation Experiment and Oceanographic top level list.

http://diu.cms.udel.edu/other/other.html

World Data Bank - Worlds coastlines, countries, rivers, etc.

ftp://sepftp.stanford.edu/pub/World_Map

WWW Database Article Archive, including information on GIS database design.

http://www.lpac.ac.uk/SEL-HPC/Articles/DBArchive.html

WWW Search engines meta-index at Centre Universitaire d'Informatique (CUI).

http://cuiwww.unige.ch/meta-index.html

WWW telnet client (only 10 users allowed at same time).

telnet://info.cern.ch

WWW Virtual Library page on Geography - maintained by Gordon Dewis.

http://hpb1.hwc.ca:10002/WWW_VL_Geography.html

WWWW - World Wide Web Worm search engine.

http://www.cs.colorado.edu/home/mcbryan/WWWW.html

Zentrum flr graphische Datenverarbeitung (ZGDV Graphic Data Centre).

http://www.igd.fhg.de

Appendix D
How to Submit and Access Data and Metadata on the USACE Node of the National Geospatial Data Clearinghouse

D-1. Introduction[1]

a. The U.S. Army Corps of Engineers (USACE) has a server on the Internet on which they store and make available to all interested persons original geospatial data and metadata created and owned by USACE agencies. It is the USACE node of the National Geospatial Data Clearinghouse. Throughout this appendix the server is called the Geospatial Server or the Server. Its network name is corpsgeo1.usace.army.mil and its Internet Protocol (IP) address is 144.3.144.20. It uses anonymous file transfer protocol (ftp), World Wide Web (WWW or Web), and limited-access ftp processes to enable users to submit and access geospatial data and metadata. This appendix is a detailed description of how to use the Server.

b. When metadata are submitted to the USACE node of the National Geospatial Data Clearinghouse, they are added to a national repository of geospatial data that is searchable and available electronically to anyone in the world with Internet access. Use of this database will eliminate duplication of data creation, enable more efficient use of resources, and make more information available for better decision making.

c. The multiple facets of the Server are described in this appendix, including the following:

(1) Process for submitting geospatial metadata to the Server.

(2) Process for submitting geospatial data to the Server.

(3) Processes to find and access data and metadata from the Server on WWW pages at http://corpsgeo1.usace.army.mil.

d. The webmaster of the Server mentioned throughout this appendix may be contacted in the following ways:

E-mail: webmaster@corpsgeo1.usace.army.mil
Phone: 603-646-4320
Fax: 603-646-4658

D-2. Responsibilities of the Geospatial Data and Systems Point of Contact

Engineer Regulation (ER) 1110-1-8156 requires that a Geospatial Data and Systems Point of Contact (GD&S POC) be designated by each Command. The GD&S POC responsibilities are defined in detail in the ER. The POC will act as the liaison between the geospatial data community of the command and Corps Headquarters, and thus will also be the POC between the Command and the Server administrators. In reference to the Server, the ER states that the GD&S POC's responsibilities include the following:

> As new data pages are developed on the USACE Clearinghouse node by HQUSACE for the Commands, GD&S POCs will review the pages and provide corrections to the Webmaster. Annually, the GD&S POC will review the Command's geospatial data pages on the USACE Clearinghouse node and forward any updates to the Webmaster.

In addition, POCs should consider the following areas of concern within their Command's geospatial data community:

a. The system(s) of filenames used to name data and metadata files by persons within their Command.

b. Records of the Server filenames that will extend beyond each employee's time of service.

c. Coordination with their information management group.

D-3. Metadata Submission to the Server

Please note that data and metadata files are currently being accepted from USACE employees who are registered to submit files on the Geospatial Data Clearinghouse Server. If you have questions about anything in this document, you can ask your GD&S POC.

a. Accessing the Server for metadata submission. (This section is the same for metadata and data submission, paragraphs D-3 and D-4.) Users need to register only once to have access to the Geospatial Clearinghouse Server for metadata and data submission. Do the following three things to register:

[1] Please note that updates to this document will be available at http://corpsgeo1.usace.army.mil/howto/.

(1) Inform the GD&S POC for your organization that you intend to submit metadata and data to the Server. You can search for the name and contact information of the GD&S POC for your organization in the GD&S POC database at URL

http://dbserver.crrel.usace.army.mil:2000/owa_crrel/owa/gds.menu

(2) To put metadata files on the USACE Geospatial Server, you must get a Corps UPASS user ID and password, and register with the UPASS system as a user on the Geospatial Metadata and Data Server. This is the same UPASS user ID and password that are recognized by the Server for data submission. (They are also the same user ID and password used in CEFMS.) The Corps standard UPASS system will be used for issuing and controlling user IDs and passwords. To get a UPASS user ID, ask your UPASS administrator for a "UPASS user ID on the Geospatial and WWW Server." To find out who your UPASS administrator is, contact your information management group.

(3) The corpsgeo1 webmaster must have your e-mail address before the metadata submission process will work for you. You must submit your name, e-mail address, and USACE organization to the Corpsgeo1 webmaster. You can do this one of three ways:

(a) Our web form at http://corpsgeo1.usace.army.mil/user-id2.html

(b) E-mail webmaster@corpsgeo1.usace.army.mil

(c) The corpsgeo1 webmaster at (603) 646-4320.

Satisfying these three requirements will enable you to put metadata and data on the Server.

b. Metadata file format: Requirements under Isite software.

(1) What are the file format requirements for metadata submission? There are three requirements for the metadata file format. Why these requirements are in place and suggestions on how to meet them are given in the following discussion. The output from CORPSMET95 conforms to these requirements. The requirements are as follows:

(a) Conformance with the Federal Geographic Data Committee (FGDC) Content Standard for Digital Geospatial Metadata (the Standard), published 8 June 1994.

(b) Indention of each line in the metadata file according to the rules required by mp, a metadata parser software program which will be explained later (Schweitzer 1997b).

(c) Plain ASCII text file format. Most word processors enable a file to be saved in ASCII text format by saving to "Text Only."

(2) Why is each of these requirements necessary? Knowing why these formats are required could mean the difference between persistence and disillusionment in the future.

(a) The Content Standard. According to Federal Executive Order 12906 and ER 1110-1-8156, metadata must be created and made available to the public for all newly developed geospatial data and for selected older geospatial data created by the agencies of the Federal government. According to the Executive Order, all USACE geospatial metadata must conform to the FGDC Content Standards for Digital Geospatial Metadata, which can be viewed at the URL

http://geochange.er.usgs.gov/pub/tools/metadata/standard/

The Content Standard defines the specific required and optional descriptors (or records) needed to describe a geospatial data set to a data user who has no previous knowledge of the data. The Overview in the Standard says,

> This standard specifies the information content of metadata for a set of digital geospatial data. The purpose of the standard is to provide a common set of terminology and definitions for concepts related to these metadata. Metadata are data about the content, quality, condition, and other characteristics of data.

The purpose for standardizing geospatial metadata is to be a part of the National Geospatial Data Clearinghouse (National Clearinghouse) and thus make the data we create accessible to the nation and the world electronically from a single source for better use of resources and better decision making. A data search initiated at the National Clearinghouse Web site can include a search of the USACE database as well as all other participating Federal agencies. Finally, a standardized format is necessary for the manipulation of the file by software, consistent use of terminology, and consistent presentation.

(b) The indentations. To function as a node of the National Geospatial Data Clearinghouse, the Server must

follow the protocol the Clearinghouse requires. The indexing software required on the National Clearinghouse Server and the USACE Server is Isite. Indexing software permits the metadata text and fields to be searchable by keyword. Isite software can index only metadata that is in Standard Generalized Markup Language (SGML) format. SGML tags will be added to your plain ASCII text-only metadata by a processing script when you put it on the Server. The indentations are needed to enable the processing software to add SGML to your file. Figure D1 shows an example of correctly indented metadata. The indentations are automatically generated by the USACE metadata creation software tool, CORPSMET95. When a metadata script is submitted to the USACE Clearinghouse Server, a program called the Sisyphus Program or SP is running to process it. The first thing that SP does is to run the metadata file through a program, mp, created by Peter Schweitzer, U.S. Geological Survey (USGS) (Schweitzer 1997b). Mp checks the metadata syntax according to the Content Standard and will output the file in SGML and Hyper Text Markup Language (HTML) format. SGML is needed for the Isite indexing software. HTML is needed to present metadata on the Web. Mp is the program that requires metadata to be in hierarchically indented format and will send back error messages if the metadata does not follow the Content Standard. Rather than require everyone submitting metadata to encode their metadata in the unfamiliar SGML, it was decided that metadata must have the hierarchical indentations.

(c) Text only. Metadata files must be in ASCII text format or they will be unreadable by SP, mp, and Isite. Word processor proprietary formats cannot be used. Files should not be in HTML or SGML format when they are submitted to the Server. No formatting codes should be embedded in the metadata file.

(3) How can I easily create metadata according to these requirements? The metadata generator software package, CORPSMET95, developed by the Corps, creates metadata in the correct format. It outputs the metadata file in the correct hierarchically indented format. It runs on Windows95 and NT operating systems. It is free of charge and menu driven. CORPSMET95 is highly recommended because of its ease of use and because the output is correctly formatted for the USACE Server. It can be downloaded from the URL

ftp://corpsgeo1.usace.army.mil/pub/corpsmet95/

This file has to be unzipped before it is installed. For more information on this process, ask your information systems group.

c. Filename conventions.

(1) When metadata files are put on the Server, a program runs that uses the filename extension of each file to determine how it will handle the file. Original metadata files should have the extension .met. Other extensions are used for

(a) Files that are to replace files currently on the Server (.rep).

(b) Files to be deleted (.del).

(c) Data files (.dat).

(2) When a file is sent via ftp to the Server, the name can have up to 100 characters, and the extension .met can be added to the file name used on the PC. All file names should consist of a continuous string of characters, which may include numbers, letters, underscore (_), hyphen (-), and dot (.). File names are case sensitive. Other special characters should be avoided, including spaces.

(3) The GD&S POCs and persons submitting metadata are responsible for keeping track of the file names of the Command's metadata. The file name will be needed if the metadata has to be replaced, updated, or deleted in the future. The file names currently in a Command's metadata directory can be seen by using anonymous ftp to view the Command's metadata directory. (See paragraph D-5 for instructions on how to use anonymous ftp.)

(4) In the past, metadata file names were changed as they were moved by SP to the Server metadata storage area. (User ID, date, and time were added to the original name.) This no longer occurs.

d. Submitting metadata to the Server. To put a metadata file on the Geospatial Server after you have registered to access the Server as described in D-3*a*, follow these steps:

(1) Plan your file name. Remember to use the .met extension for metadata file submissions. You can submit multiple files during one ftp session, including metadata and data files.

(2) Use your computer's ftp utility. There are many variations of ftp software. It would be impossible to describe them all. For assistance with the ftp utility on your computer, ask your information management group.

```
Identification_Information:
    Citation:
        Citation_Information:
            Originator: New Orleans District, U.S. Army Corps of Engineers
            Publication_Date: 19941206
            Title: Mississippi River Southwest Pass Hydrographic Surveys of 19941206
            Edition: N/A
            Geospatial_Data_Presentation_Form: map
            Publication_Information:
                Publication_Place: New Orleans, LA
                Publisher: New Orleans District, U.S. Army Corps of Engineers
            Online_Linkage: http://www.lmn.usace.army.mil/ops/odt/nav-cond.htm
    Description:
        Abstract:
            The Mississippi River Southwest Hydrographic
            Surveys are compiled in order to monitor and
            maintain  Mississippi River Channel conditions at
            South and Southwest Passes and Pass A Loutre.
        Purpose:
            The purpose of the Mississippi River
            Southwest Pass Hydrographic Surveys is to
            provide a current survey and map of the Mississippi River
            for the purpose of recording changes in channel
            conditions, secondarily as an aid to navigation,
            and to be used as an engineering and planning tool for
            future Flood Control, Navigation, and Hurricane Protection
            projects.
    Time_Period_of_Content:
        Time_Period_Information:
            Single_Date/Time:
                Calendar_Date: 19941206
        Currentness_Reference: Publication Date
    Status:
        Progress: In work
        Maintenance_and_Update_Frequency: Daily
    Spatial_Domain:
        Bounding_Coordinates:
            West_Bounding_Coordinate: -089.300000
            East_Bounding_Coordinate: -089.070000
            North_Bounding_Coordinate: +29.100000
            South_Bounding_Coordinate: +28.530000
    Keywords:
        Theme:
            Theme_Keyword_Thesaurus: Tri - Service Spatial Data Standard
                Theme_Keyword: Boundaries
                Theme_Keyword: Hydrography
```
[Note: Metadata file cropped here.]

Figure D-1. An example of correctly indented metadata. The original metadata file was created by the New Orleans District using CORPSMET95. Only a portion of the entire metadata file is shown here.

(3) Use the following parameters to ftp to your home directory on corpsgeo1:

Host: corpsgeo1.usace.army.mil
Login ID: *yourUPASSid*
Password: *yourUPASSpassword*

When you have successfully logged on to corpsgeo1, you will be in your home directory. This is the only place on the Server where you can put files.

(4) You are now ready to submit the file to the Server. (The metadata file must be ASCII text, and must be sent in ASCII text mode.) In a command line view on a Windows3.x/Win95/NT or UNIX operating system, the command to send the file to the Server is

put filename.met

(5) When you are done putting metadata on the Server, close the session and exit from the ftp software. If you stop communicating with the ftp site for some period of time during the ftp session, you may be timed out; i.e., you will not be connected to the Server anymore. If this occurs, you will have to ftp to the Server again.

(6) SP will process new files approximately once an hour. If a metadata submission is successfully processed, you will receive an e-mail message within a few hours saying so. If the file is not successfully processed, you will receive an e-mail message within a few hours stating this and explaining why it was not successfully processed.

e. Replacing or deleting metadata files. To replace or delete a metadata file on the Server, you will need to know the exact file name of the file to be replaced or deleted. You can use anonymous ftp or use the World Wide Web and go to the URL ftp://corpsgeo1.usace.army.mil to go to your organization's metadata directory to look for the file name you need. Login procedures and the directory structure for anonymous ftp on the corpsgeo1 site are given in paragraph D-5. If you are trying to replace or delete files that you did not put on the Server, you will have to contact the webmaster with proof that you are now responsible for these files.

(1) To replace (or update) a metadata file.

(a) Have the replacement file ready on your local computer. The file name of the replacement file should be in the format

filename(.met).rep

where filename(.met) is the name of the file to be replaced. The original file name will end with .met so the file name you type here will end with .met.rep. The extension .rep will cause the Server to replace the metadata file currently in your Command's metadata directory having the name filename.met with the file you just put in your home directory.

(b) Ftp to the Server. Use the following parameters to ftp to your home directory on corpsgeo1:

Host: corpsgeo1.usace.army.mil
Login ID: *yourUPASSid*
Password: *yourUPASSpassword*

(c) You are now ready to submit the file to the Server. (The metadata file must be ASCII text, and must be sent in ASCII text mode.) In a command line view on a Windows3.x/Win95/NT or UNIX operating system, the command to send the file to the Server is

put filename.met.rep

(d) When you are done putting metadata on the Server, close the session and exit from the ftp software.

(2) To delete a metadata file.

(a) First create a dummy file on your local computer. It does not matter what is in the dummy file. The file name should be in the format

filename(.met).del

where filename(.met) is the name of the dummy file you created on your local computer and the name of the file to be deleted. The file name of the existing metadata file will end with .met, so the name of the dummy file will end with .met.del. The extension .del will cause the Server to delete the metadata file you have named.

(b) Ftp to the Server. Use the following parameters to ftp to your home directory on corpsgeo1:

Host: corpsgeo1.usace.army.mil
Login ID: *yourUPASSid*
Password: *yourUPASSpassword*

(c) You are now ready to submit the dummy file to the Server. (The metadata file must be ASCII text, and must be sent in ASCII text mode.) In a command line view on a Windows3.x/Win95/NT or UNIX operating system, the command to send the file to the Server is

put filename.met.del

(d) When you are done putting metadata on the Server, close the session and exit from the ftp software.

f. Editing collection metadata files. The USACE geospatial data and metadata available on the Web include *collection metadata* and *detailed metadata.* A collection metadata file is a single metadata file that describes a series of related data files that are routinely collected by an agency. For example, the data may be hydrographic files that have been collected for 20 years. A detailed metadata file describes a unique data file or a collection of data files that are not routinely collected by an agency. The corpsgeo1 team has created a series of templates of collection metadata for USACE Commands to edit or delete, as appropriate. Links to these are found under each Command's data pages. The changes to these pages can be made on a paper copy or electronic copy of the file. Electronic editing is preferred. You can make the changes to the electronic file as described below, then send the resulting file to the webmaster as an e-mail attachment.

(1) Electronic editing

(a) While browsers differ, general instructions for saving an electronic copy of a Web page to your local computer are given here. To download the Web page to your PC, use your browser to view the collection metadata file Web page. Save the file to your PC by using the browser menu options: File/Save as. Choose to save in the format type HTML or text. Note where in your directory structure the file is being saved and its file name. Press enter.

(b) When it is saved, the collection metadata file will have all the HTML tags necessary for viewing it from your Web browser on your local PC. To view the file on your PC using your browser, use the browser menu options File/Open File in Browser or File/Open/Browse to locate and open the electronic copy of the metadata file you just downloaded.

(c) You can then work in your word processor to make needed changes in the file and view the results from your browser. Please remember to open and save file in "Text Only" format in your word processing software. Any images in these files will not appear as you edit and view them on your local computer. This is normal. When the file is placed back on the Server, the images will appear as they should.

(2) Editing a paper copy. Print a hard copy of the page from your web browser while you are viewing it. Edit it and fax the edited version to the corpsgeo1 webmaster at 603-646-4658. Please make your edits clearly. Please let the webmaster know the fax is coming.

D-4. Data Submission to the Server

Please note that data and metadata files are currently being accepted on the Corps Geospatial Server from USACE employees who are registered to use the Server. The producers of geospatial data are responsible for the quality, integrity, and maintenance of the data that they produce.

a. Accessing the Server for data submission. The process for registering to use the Server for metadata and data submission is exactly the same. The three requirements are described in paragraph D-3*a*.

b. Data file formats. Data in any format will be accepted on the Server. This includes output generated by any Computer-Aided Design and Drafting (CADD) system or Geographic Information System (GIS). It includes any data that contain geospatial coordinates. Please be aware of the file name conventions for placing data on the Server as described below. When you are sending binary data, remember to set the mode for binary. This may be done automatically by the ftp software.

c. Filename conventions.

(1) A UNIX script called SP, or the Sisyphus Program, is running on the Server at all times. This program scans the new files in your directory and uses the filename extension of each file to determine how it will handle the file. Original data files should have the extension .dat. Other extensions are used for

(a) Files that are to replace files currently on the Server (.rep).

(b) Files to be deleted (.del).

(c) Metadata files (.met).

(2) When a file is sent to the Server, the name can have up to 100 characters, and the extension.dat can be added to the filename used on the PC. For example, an Arc Info export file could be called white_riverVT19970607.e00.dat. If your local computer does not allow long file names, you *

Table B-4. (Continued)

Joint Alteration Number	(J_a)	ϕ_r (approx.)
(a) Rock wall contact		
A. Tightly healed, hard, nonsoftening, impermeable filling i.e. quartz or epidote...........................	0.75	(-)
B. Unaltered joint walls, surface staining only......................	1.0	(25°-35°)
C. Slightly altered joint walls. Non-softening mineral coatings, sandy particles, clay-free disintegrated rock etc..........................	2.0	(25°-30°)
D. Silty-, or sandy-clay coatings, small clay-fraction (non-softening)	3.0	(20°-25°)
E. Softening or low friction clay mineral coatings, i.e. kaolinite, mica. Also chlorite, talc, gypsum and graphite etc., and small quantities of swelling clays. (Discontinuous coatings, 1-2 mm or less in thickness).................	4.0	(8°-16°)
(b) Rock wall contact before 10 cms shear		
F. Sandy particles, clay-free disintegrated rock etc.............	4.0	(25°-30°)
G. Strongly over-consolidated, non-softening clay mineral fillings (Continuous, <5 mm in thicknes)....	6.0	(16°-24°)
H. Medium or low over-consolidation, softening, clay mineral fillings. (continuous, <5 mm in thickness)...	8.0	(12°-16°)
J. Swelling clay fillings, i.e. montmorillonite (Continuous, <5 mm in thicknes). Value of J_a depends on percent of swelling clay-size particles, and access to water etc......................	8.0-12.0	(6°-12°)
(c) No rock wall contact when sheared		
K., L., M. Zones or bands of disintegrated or crushed rock and clay (see G., H., J. for description of clay condition).........................	6.0, 8.0 or 8.0-12.0	(6°-24°)
N. Zones or bands of silty- or sandy clay, small clay fraction (nonsoftening)....................	5.0	
O., P., R. Thick, continuous zones or bands of clay (see G., H., J. for description of clay condition).....	10.0, 13.0 or 13.0-20.0	(6°-24°)

Note:

(i) Values of $(\phi)_r$ are intended as an approximate guide to the mineralogical properties of the alteration products, if present.

(Continued)

Table B-4. (Concluded)

Stress Reduction Factor

				(SRF)	Note
	(a) Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated.				(i) Reduce these values of SRF by 25-50% if the relevant shear zones only influence but do not intersect the excavation.
A.	Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)			10.0	
B.	Single weakness zones containing clay, or chemically disintegrated rock (depth of excavation ≤50 m)			5.0	
C.	Single, weakness zones containing clay, or chemically disintegrated rock (depth of excavation >50 m)			2.5	
D.	Multiple shear zones in competent rock (clay free), loose surrounding rock (any depth)			7.5	
E.	Single shear zones in competent rock (clay free) (depth of excavation ≤50 m)			5.0	
F.	Single shear zones in competent rock (clay free) (depth of excavation >50 m)			2.5	
G.	Loose open joints, heavily jointed or "sugar cube" etc. (any depth)			5.0	
	(b) Competent rock, rock stress problems.	σ_c/σ_1	σ_t/σ_1		
H.	Low stress, near surface	>200	>13	2.5	(ii) For strongly anisotropic stress field (if measured): when $5 \le \sigma_1/\sigma_3 \le 10$, reduce σ_c and σ_t to $0.8\ \sigma_c$ and $0.8\ \sigma_t$; when $\sigma_1/\sigma_3 > 10$, reduce σ_c and σ_t to $0.6\ \sigma_c$ and $0.6\ \sigma_t$ where: σ_c = unconfined compression strength, σ_t = tensile strength (point load), σ_1 and σ_3 = major and minor principal stresses.
J.	Medium stress	200-10	13-0.66	1.0	
K.	High stress, very tight structure (Usually favorable to stability, may be unfavorable to wall stability)	10-5	0.66-0.33	0.5-2.0	
L.	Mild rock burst (massive rock)	5-2.5	0.33-0.16	5-10	
M.	Heavy rock burst (massive rock)	<2.5	<0.16	10-20	
	(c) Squeezing rock; plastic flow of incompetent rock under the influence of high rock pressures.				
N.	Mild squeezing rock pressure			5-10	
O.	Heavy squeezing rock pressure			10-20	(iii) Few case records available where depth of crown below surface is less than span width. Suggest SRF increase from 2.5 to 5 for such cases (see H).
	(d) Swelling rock; chemical swelling activity depending on presence of water				
P.	Mild swelling rock pressure			5-10	
R.	Heavy swelling rock pressure			10-15	

Joint Water Reduction Factor

		(J_w)	Approx. water pressure (kg/cm^2)	Note
A.	Dry excavations or minor inflow, i.e. 5 l/min. locally	1.0	<1	(i) Factors C to F are crude estimates. Increase J_w if drainage measures are installed.
B.	Medium inflow or pressure occasional outwash of joint fillings	0.66	1.0-2.5	(ii) Special problems caused by ice formation are not considered.
C.	Large inflow or high pressure in competent rock with unfilled joints	0.5	2.5-10.0	
D.	Large inflow or high pressure, considerable outwash of joint fillings	0.33	2.5-10.0	
E.	Exceptionally high inflow or water pressure at blasting, decaying with time	0.2-0.1	>10.0	
F.	Exceptionally high inflow or water pressure continuing without noticeable decay	0.1-0.05	>10.0	

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DEPARTMENT OF THE ARMY
US Army Corps of Engineers
Washington, DC 20314-1000

EM 1110-1-2909
Change 2

Engineer Manual
No. 1110-1-2909

1 July 1998

Engineering and Design
GEOSPATIAL DATA AND SYSTEMS

1. This change 2 to EM 1110-1-2909, 1 Aug 96, adds Chapter 11, Accuracy Standards for Engineering, Construction, and Facility Management Surveying and Mapping.

2. Substitute the attached pages as shown below:

Chapter	Remove Page	Insert Page
Contents	i-ii	i-iv 11-1 thru 11-31

3. File this change sheet in front of the publication for reference purposes.

4. This change is approved for public release. Distribution is unlimited.

FOR THE COMMANDER:

ALBERT J. GENETTI, JR.
Major General, USA
Chief of Staff

DEPARTMENT OF THE ARMY
U.S. Army Corps of Engineers
Washington, DC 20314-1000

EM 1110-1-2909
Change 1

CECW-EP

Engineer Manual
No. 1110-1-2909

30 April 1998

Engineer and Design
GEOSPATIAL DATA AND SYSTEMS

1. This Change 1 to EM 1110-1-2909, 1 Aug 96, adds instructions on how to submit and access data and metadata on the USACE node of the National Geospatial Data Clearinghouse, Appendix D.

2. Substitute the attached pages as shown below:

Chapter	Remove page	Insert page
Contents	ii	ii D-1 thru D-11

3. File this change sheet in front of the publication for reference purposes.

FOR THE COMMANDER:

ALBERT J. GENETTI, JR.
Major General, USA
Chief of Staff

DEPARTMENT OF THE ARMY
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EM 1110-1-2909

CECW-EP

Engineer Manual
No. 1110-1-2909

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Engineering and Design
GEOSPATIAL DATA AND SYSTEMS

1. Purpose. This manual provides detailed technical guidance and procedures for compliance with the policy in Engineer Regulation (ER) 1110-1-8156, regarding Geospatial Data and Systems (GD&S) within the U.S. Army Corps of Engineers (USACE).

2. Applicability. This manual applies to all USACE Commands having civil works, military construction, and environmental restoration responsibilities. This manual specifically applies to functional areas having responsibility for regulatory investigations and studies, planning studies, real estate, emergency operations, and other functions involving automated geospatial data systems for surveying, mapping, or geospatial database development, such as modeling, and to GD&S that are used to produce a variety of products including: river and harbor maps, charts, and drawings; real estate tract or parcel maps; small- and medium-scale engineering drawings; survey reports; environmental studies; hazardous, toxic, and radioactive waste studies; and channel condition reports. This manual applies to' in-house and contracted efforts.

3. Discussion. ER 1110-1-8156 establishes general criteria and presents policy and guidance for the acquisition, processing, storage, distribution, and utilization of non-tactical geospatial data throughout USACE. This policy is in compliance with Executive Order 12906, Coordinating Geographic Data Acquisition and Access: The National Spatial Data Infrastructure (NSDI) and other appropriate standards, including the Spatial Data Transfer Standard/Federal Information Processing Standard 173, Federal Geographic Data Committee standards, and T&Service Spatial Data Standards. This manual also provides detailed technical guidance and requirements and identifies standards for GD&S. By using this manual, USACE will maximize its use of GD&S technologies; will promote interoperability among GD&S technologies; will reduce duplication of geospatial data collection and software development; will support the digital geospatial data life cycle; and will strengthen the USACE role in the NSDI.

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Engineering and Design
GEOSPATIAL DATA AND SYSTEMS
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Chapter 1
Introduction

1-1. Purpose

This manual provides detailed technical guidance and procedures for compliance with the policy in Engineer Regulation (ER) 1110-1-8156. That regulation establishes general criteria and presents policy and guidance for the acquisition, processing, storage, distribution, and utilization of non-tactical geospatial data throughout the U.S. Army Corps of Engineers (USACE) and in compliance with Executive Order (EO) 12906, Coordinating Geographic Data Acquisition and Access: The National Spatial Data Infrastructure (NSDI) and other appropriate standards, including the Spatial Data Transfer Standard (SDTS)/Federal Information Processing Standard (FIPS) 173, Federal Geographic Data Committee (FGDC) standards, and Tri- Service Spatial Data Standards (TSSDS). This manual also provides detailed technical guidance and requirements and identifies standards for Geospatial Data and Systems (GD&S). By using this manual, USACE will maximize its use of GD&S technologies; will promote interoperability among GD&S technologies; will reduce duplication of geospatial data collection and software development; will support the digital geospatial data life cycle; and will strengthen the USACE role in the NSDI.

1-2. Applicability

This manual applies to all USACE Commands having civil works, military construction, and environmental restoration responsibilities. This manual specifically applies to functional areas having responsibility for regulatory investigations and studies, planning studies, real estate, emergency operations, and other functions involving automated GDS for surveying, mapping, or geospatial database development, such as modeling, and to GD&S that are used to produce a variety of products including: river and harbor maps, charts, and drawings; real estate tract or parcel maps; small- and medium-scale engineering drawings; survey reports; environmental studies; hazardous, toxic, and radioactive waste (HTRW) studies; and channel condition reports. This manual applies to in-house and contracted efforts. USACE customers for reimbursable work who are required to comply with the EO 12906, such as the Department of Defense (DoD) installations, Environmental Protection Agency, and the Federal Emergency Management Agency, will determine their level of compliance. These customers may opt to incorporate compliance with the EO into contracts with USACE or may accomplish compliance unassisted by USACE.

1-3. References

Required and related publications are listed in Appendix A.

1-4. Abbreviations and Acronyms

Abbreviations and acronyms used in this publication are listed in Appendix B.

1-5. Definitions

a. Geospatial Data - non-tactical data referenced, either directly or indirectly, to a location on the earth.

b. Geospatial Data Systems (GDS) - any automated system that employs geospatial data including Geographic Information Systems (GIS), Land Information Systems (LIS), Remote Sensing or Image Processing Systems, Computer-Aided Design and Drafting (CADD) systems, Automated Mapping/Facilities Management (AM/FM) systems, and other computer systems that employ or reference data using either absolute, relative, or assumed coordinates such as hydrographic surveying systems.

c. Geospatial Data and Systems (GD&S) - Geospatial data and the GDS that create and process the data.

d. USACE Commands - all subordinate entities of the U.S. Army Corps of Engineers including districts, divisions, research laboratories, and field offices.

e. Metadata - descriptive information about the data. Metadata describes the content, quality, fitness for use, access instructions, and other characteristics about the geospatial data.

f. National Geospatial Data Clearinghouse (Clearinghouse) - a distributed, electronic network of geospatial data producers, managers, and users operating on the Internet. The Clearinghouse is a key element of EO 12906 and will allow its users to determine what geospatial data exist, find the data they need, evaluate the usefulness of the data for their applications, and obtain or order the data as economically as possible.

g. USACE Clearinghouse Node - HQUSACE established and maintains a computer network server on the National Geospatial Data Clearinghouse. This node functions as the primary point of public entry to the USACE geospatial data discovery path in the Clearinghouse. A separate electronic data page for each USACE Command has been established on the server. The Internet Universal Resource Locator (URL) address for the USACE Clearinghouse node is http://corps_geo1.usace.army.mil

1-6. Scope

This manual provides detailed technical guidance and requirements and identifies standards related to GD&S. This includes: requirements analysis; implementation plans; organizational issues such as staffing, training, and managerial support; system configuration and procurement of hardware, software, and telecommunications equipment; standards; data issues such as collection and acquisition, metadata (documentation), schemas (classification), and electronic clearinghouses (data locator and access service); applications; and evaluation criteria. Implementation actions are included in paragraph 7 of ER 1110-1-8156.

1-7. Exclusions

a. Spatial data and computer systems that do not use coordinates that are directly or indirectly referenced to a position on the Earth are not required to adhere to this regulation. This exempts architectural, mechanical, electrical, structural, and sanitary engineering data and drawings of objects typically inside buildings, as well as the CADD systems used to develop such data and drawings.

b. Site plans showing building structure footprints or any data set about features on the exterior of a building or structure are geospatial data, are compatible wtih the NSDI concept, and are not excluded from this regulation. This data may use relative, assumed, or geographic coordinates and may be stored in a CADD or GIS environment.

c. This regulation also excludes business systems, such as those that focus on textual and statistical information that is created, stored, manipulated, queried, displayed, and transferred differently than geospatial data.

d. This regulation also excludes tactical spatial data and associated computer systems such as those used for fire control, targeting, and mission planning.

e. Users of excluded systems may find this document useful in implementing, organizing or managing their particular type of automated system; in identifying applicable standards; or in creating and maintaining a database. This manual may enhance interoperability among GDS and other data systems, and users of all automated systems are encouraged to coordinate, when appropriate, with users, managers, and administrators of other automated systems.

1-8. Brand Names

The citation in this manual of brand names of commercially available products does not constitute official endorsement or approval of the use of such products.

Chapter 2
GD&S Applications in USACE

2-1. Introduction

a. This chapter is intended to provide the reader with a feel for the variety of Geospatial Data and Systems (GD&S) applications in USACE. This chapter begins with the project development process in USACE as it relates to GD&S. The types of GD&S projects at the District, Division, Headquarters, and Laboratory levels are then discussed. Information on the many application areas and the Districts where they occur is presented, along with several typical and atypical sample applications.

b. A Geospatial Data System (GDS) has been defined as the computer hardware and software used to input, store, retrieve, manipulate, analyze, and plot/print geospatially (geographically) referenced digital data, referred to as Geospatial Data. Collectively these are referred to as GD&S for Geospatial Data and Geospatial Data System. GD&S is one of the fastest growing technologies and has changed the way USACE operates. GD&S technology provides a tool for planners, resource managers, engineers, scientists, real estate specialists, and others to perform complex analyses faster and more reliably than ever before. More types of data can be archived, retrieved and studied, making it possible to examine both new and old problems in a more comprehensive way. Even hardcopy products, such as maps and photographs, can be scanned or features can be extracted to allow more advanced processing.

c. Data sharing capability among geospatial data users in a geographical region is important. USACE Districts and Divisions may be called upon to share geospatial data with other Federal agencies, State and Local governments, and municipalities more often than with other USACE commands. Also, billions of dollars are spent annually by federal, state and local governments, and industry developing databases that could be used by USACE. The ability to incorporate geospatial data developed by organizations is vital if USACE is to cost-effectively and rapidly implement GD&S technology.

d. USACE has a great diversity of GD&S applications including Wetlands Permitting and Analysis, Environmental Restoration, Resource Management, Habitat Analyses, Environmental Change Detection, Aquatic Plant Tracking, Historical Preservation, Hydrology and Hydraulics, Channel/Inland Waterways Maintenance, Emergency Response, Flood Plain Mapping, Real Estate/Cadastral, Master Planning, District/Construction Management, Socio-economic Analysis, and Geologic/Geomorphic Analysis. These applications support both the USACE civil and military missions. These applications emphasize providing access to geospatial data and rendering the data into information through: (1) quantitative and qualitative analyses, and (2) visual products.

2-2. The GD&S Development Process in USACE

GD&S technology is an enabler that may belong in many offices. It should not be viewed as exclusive to one portion of a USACE Command, but should be diffused throughout the Command to those functions where it is useful.

2-3. USACE GD&S Application Categories

a. District GD&S Application Categories. GD&S at the District level is employed for geospatial data analysis in support of engineering projects. Numerous District level data sets are geospatial in nature and are best accessed and managed by using GD&S technologies. Among the means of access are visualization, spatial query and geospatial data layer integration. These technologies support basic analysis and can provide modeling support. The result is a focusing of resources to support both quantitative and qualitative decision making in the District mission areas and the preparation of decision support materials for the Division and Headquarters.

b. Division and Headquarters GD&S Application Categories. GD&S at the Division and Headquarters level is not one of analysis but the visualization and display of District geospatial analytical products for USACE-wide or corporate decisions. This is essentially an Executive Briefing System.

c. Laboratory GD&S Application Categories. GD&S at the laboratory level is complex with many unique analysis and modeling applications in a variety of advanced research areas and for project support to districts. The results are again subject to review at Headquarters through visualization and query. Laboratory efforts are also directed to the development of configuration managed geospatial applications for the civil side of USACE. Advanced GD&S projects at USACE Laboratories include terrain visualization, modeling and simulation of environmental phenomena, hyper-spectral analysis of imagery to support change detection, and applications research.

d. Sample GD&S. There are many application areas for GD&S in USACE. A typical application is real estate land records management. Tract data can be entered into a GD&S for all USACE-owned lands. Attribute data includes values such as parcel numbers, ownership status, dates when interests were acquired and easements. Accuracy of data is best maintained at the cadastral level to support real estate

management [North Central Division Implementation Plan, 1989].

Occasionally there are atypical applications as well. These are applications which are unique to a particular USACE Command because of its unique mission or capabilities. One example is the prediction of ice jam flooding by the Cold Regions Research and Engineering Laboratory (CRREL).

Chapter 3
GD&S Organizational Issues

3-1. Organizational Model for GD&S Usage Within USACE

This chapter describes an organizational model for GD&S usage within USACE and can be used as a guide in determining the level of GD&S functionality and responsibility that is appropriate for each Command. The exchange of geospatial products and information is to be accomplished among all USACE Commands using the Internet.

a. Districts. The Districts are to have fully functional geospatial data systems to meet project needs and mission requirements. This includes the GD&S necessary for data collection and database creation, geospatial analysis, and product generation. Data collected, acquired, or created at the District is to be documented (metadata) and maintained at the District unless it is developed for and delivered to a customer. The District shall make the metadata for in-house data sets available to the National Geospatial Data Clearinghouse, where appropriate. An agreement should be reached with a customer, prior to beginning a project, as to who is responsible for providing the metadata to the Clearinghouse.

GD&S activities will normally be distributed across the various functions in the District. Each function -- Planning, Real Estate, Operations, etc. -- will operate a GDS that meets their particular requirements. Centralized oversight and technical committees will provide the structure necessary to eliminate redundant data acquisition and improve efficiency by providing guidance on training and hardware and software purchases and through the sharing of experiences and expertise.

b. Divisions. The Divisions typically require less geospatial data system functionality than the Districts. The functionality required is that which will allow Division staff to view, using commercially available tools such as ArcView, geospatial data products created by the Districts so they can make management decisions, conduct executive briefings, maintain an overview of the Division activity, and coordinate GD&S activities within the Division. The Division shall make the metadata for any in-house data sets available to either the National Geospatial Data Clearinghouse where appropriate.

c. Research and Development Laboratories. The Research and Development (R&D) Laboratories require complete geospatial data systems functionality to meet their research and customer needs. The R&D Laboratories need to be able to advise the field on GD&S technologies, provide project support where project GD&S environments vary, evaluate new GD&S technologies, and develop algorithms and designs for systems to be fielded. The choice of GD&S must be kept flexible in the R&D Laboratory environment. Data collected, acquired, or created at the Laboratory is to be documented and maintained at the Laboratory unless it is delivered to another USACE Command or customer. The Laboratory shall make the metadata for in-house data sets available to the National Geospatial Data Clearinghouse, where appropriate. An agreement should be reached with a customer, prior to beginning the project, as to who is responsible for providing the metadata to the Clearinghouse

d. Headquarters. Headquarters typically requires less geospatial data system functionality than the Districts, Divisions, or Laboratories. The functionality required is that which will allow HQUSACE staff to view, using a commercially available tool such as ArcView or VistaMap, geospatial data products created by the field so they can make management decisions, conduct executive briefings, maintain an overview of and coordinate USACE GD&S activity.

3-2. Location of GD&S Within a Command

The location of GD&S within a USACE Command is a determination to be made at the Command level. The considerations are: (1) the current and future GDS functionality and data requirements, (2) the sources of support and funding within the Command, and (3) the location of GD&S expertise within the Command. In most cases, a close linkage to the individual divisions will be preferable because geospatial data exploitation is just one of many enabling technologies supporting the Command missions. The establishment of the oversight and technical committees and informal users groups will aid divisions that are acquiring GD&S technology for the first time by providing expertise and experience.

3-3. HQUSACE Geospatial Data and Systems Manager

The Chief, Engineering Division, Directorate of Civil Works, HQUSACE (CECW-E) will serve as the USACE GD&S Manager and will represent Civil Works on the Tri-Service CADD/GIS Technology Center Executive Steering Group and will represent DoD facilities, civil works, and environmental interests on the FGDC.

3-4. HQUSACE Geospatial Data and Systems Coordination Committee

The HQUSACE GD&S Coordination Committee is chaired

by CECW-EP-S, composed of HQUSACE personnel who play a role in GD&S, and addresses GD&S issues from a corporate perspective. Each participating HQUSACE Directorate, such as Information Management, Real Estate, Civil Works, Military Programs, and Research and Development, will nominate a member to this committee. The Coordination Committee will meet at least twice per year. The chair will support the USACE GD&S Manager, will grant waivers of compliance for ER 1110-1-8156, will review information copies of Command GD&S implementation plans and evaluation reports, and will consider funding GD&S Field Advisory Group recommendations and other corporate GD&S activities.

3-5. USACE Geospatial Data and Systems Field Advisory Group

The USACE Geospatial Data and Systems (GD&S) Field Advisory Group assists HQUSACE in defining the role of the Corps of Engineers in the National Spatial Data Infrastructure and also recommends implementations of geospatial data standards and related technologies within USACE. The GD&S Field Advisory Group is composed of approximately one representative from a District in each Division and one from each USACE R&D Laboratory. The members are selected by CECW-EP-S based on their expertise in GD&S technologies and applications. They usually meet twice per year and they elect the chair.

3-6. USACE Commanders.

Commanders have two actions per ER 1110-1-8156. They will appoint a GD&S Point of Contact (POC) to act as a liaison between the command and HQUSACE/CECW-EP-S. Beginning with the FY97 Civil Works budget cycle, USACE Commanders will certify that their Command has accessed the Clearinghouse, contributed metadata to the Clearinghouse, determined via the Clearinghouse that needed geospatial data are not available from an existing source, and that possible data collection partnerships have been explored. This certification, included as Appendix B in ER 1110-1-8156, will be submitted to USACE annually as part of the Civil Works Budget submittal.

3-7. Geospatial Data and Systems Oversight Committee

The Geospatial Data and Systems (GD&S) Oversight Committee is a requirement of ER 1110-1-8156. The purpose of this committee is to promote interoperability among the various GD&S efforts within the USACE Command from a corporate or administrative perspective and to approve the plans and procedures drafted by the GD&S Technical Committee for complying with this manual and ER 1110-1-8156. The oversight committee will address funding, administrative, and policy issues in coordination with the commander and executive board as necessary.

The GD&S Oversight Committee consists of the chief, or a designated representative, of any division or office within the USACE Command that has an interest in geospatial data. This includes, but is not limited to, those working in the areas of planning, environmental analysis, project management, aerial photography and remote sensing, information management, waste water control, water quality analysis, emergency management, engineering design, facility management, real estate, regulatory functions, geotechnical analysis, hydrographic and land surveying, terrain analysis, economic analysis, and forestry. The final composition of the Committee, including the chair, is defined by the Command. Meeting schedules and procedures are to be developed by the Command. However, they should meet at least twice a year and as necessary to react to actions of the GD&S Technical Committee. If a Command has a pre-established group that performs functions similar to or a superset of the Oversight Committee, it may choose to continue with that group as long as the functions described in this EM and ER 1110-1-8156 are performed effectively.

The responsibilities of the GD&S Oversight Committee are:

- Review and approve the GD&S Implementation Plan (IP) developed by the GD&S Technical Committee

- Forward an information copy of the initial or base IP to HQUSACE, as well as subsequent IP revisions which are created at least every three years.

- Review the GD&S Performance Evaluation submitted annually by the GD&S Technical Committee. Forward an information copy of the Evaluation to HQUSACE/CECW-EP-S.

- Ensure that the Command documents new geospatial data (data created after January 1995) using the FGDC Content Standard for Digital Geospatial Metadata.

- Ensure that the Command documents existing (pre-Janaury 1995) geospatial data to the extent practicle.

- Ensure that the Command submits metadata to the Clearinghouse.

- Ensure that the Command Utilizes the Clearinghouse prior to spending Federal funds on data collection or creation, to determine if the required data already exists.

- Ensure that the Command provides public access to geospatial data.

3-8. Geospatial Data and Systems Technical Committee

a. The Geospatial Data and Systems (GD&S) Technical Committee is a requirement of ER 1110-1-8156. The purpose of the GD&S Technical Committee is to promote interoperability among the various GD&S efforts within the USACE Command from a technical perspective and to ensure adherence to this manual and ER 1110-1-8156, also from a technical perspective. If a Command has a pre-established group that performs functions similar to or a superset of the Technical Committee, it may choose to continue with that group as long as the functions described in this EM and ER 1110-1-8156 are performed effectively.

b. The GD&S Technical Committee is comprised of members selected from all persons responsible for geospatial data management and other interested persons in the USACE Command. This includes, but is not limited to, those working in the areas of planning, environmental analysis, project management, aerial photography and remote sensing, information management, waste water control, water quality analysis, emergency management, engineering design, facility management, real estate, regulatory functions, geotechnical analysis, hydrographic and land surveying, terrain analysis, economic analysis, and forestry. The final composition of the Committee, including the chair, is defined by the Command. The meetings should be held in an open forum with a published agenda. An invitation for all interested parties to attend the meetings of the Technical Committee is encouraged.

c. The GD&S Technical Committee shall meet at least four (4) times per year to discuss GD&S activities within the USACE Command. The Committee shall keep everyone in the Command, including personnel at the field offices, informed of the GD&S efforts in the Command. It is recommended that the chair of the Technical Committee be rotated annually.

d. The responsibilities of the GD&S Technical Committee are:

- Prepare the initial GD&S Implementation Plan and forward it to the GD&S Oversight Committee for approval.

- Evaluate the execution of the GD&S Implementation Plan annually to determine if the execution is on schedule and moving in the direction set by the GD&S Implementation Plan. Forward this evaluation to the GD&S Oversight Committee.

- Review the GD&S Implementation Plan itself every three years, revise it as necessary and forward it to the GD&S Oversight Committee for review and approval.

- Ensure that the Command documents new geospatial data (data created after January 1995) using the FGDC Content Standard for Digital Geospatial Metadata.

- Ensure that the Command documents existing (pre-Janaury 1995) geospatial data to the extent practicle.

- Ensure that the Command submits metadata to the Clearinghouse.

- Ensure that the Command Utilizes the Clearinghouse prior to spending Federal funds on data collection or creation, to determine if the required data already exists.

- Ensure that the Command provides public access to geospatial data.

- Establish priorities for and coordinate data acquisition and development within the organization.

- Appoint representatives, where appropriate to the organization's mission, to coordinate with local, state, and National GD&S Technical Committees and Task Forces.

3-9. Geospatial Data and Systems Point of Contact

The GD&S Point of Contact (POC) is a requirement of ER 1110-1-8156. Each USACE Commander shall appoint an individual to act as the GD&S POC. The selection of the GD&S POC is an internal organizational decision. The GD&S POC may be drawn from any area of geospatial data expertise including, but not limited to, GIS, remote sensing, surveying and cartography, or CADD. The GD&S POC has the responsibility to disseminate relevant information to all members of the Command's geospatial data community, including field offices and will be the focal point for information exchange with HQUSACE. The GD&S POC will sit on the GD&S Technical Committee and may be, but is not required to be, the chairperson. The GD&S POC will also be an advisor to the GD&S Oversight Committee.

As new data pages are developed by HQUSACE on the USACE node of the National Geospatial Data Clearinghouse GD&S POCs will review their Command's pages and provide corrections to the Webmaster. Annually, the GD&S

POC will review his Command's geospatial data pages on the USACE Clearinghouse node and forward any updates to the Webmaster.

3-10 GD&S Users Groups and Special Interest Groups

The Technical Committee may choose to form one or more GD&S users groups or special interest groups. These less formal groups, comprised primarily of technical experts, have been found to be effective at addressing specific GD&S issues. Users/special interest groups may be established to address functional areas, e.g., a real estate users group; technology areas, e.g., an Internet special interest group; or standards issues, e.g., the data interchange users group. These groups should keep the GD&S Technical Committee informed of the activities and may be tasked with focused studies, as needed.

3-11. Management Issues

a. Once a GD&S Implementation Plan is in place, a number of management issues will arise. A critical one is the need for upper management support. By their approval of the Implementation Plan, the GD&S Oversight Committee has provided middle management approval and support.

b. USACE Command management needs to address funding to implement the Implementation Plan. This includes funding for the initial GD&S and geospatial data and funding for operations and maintenance (O&M). These O&M costs may need to be allocated to projects or may be charged to Command overhead. Funding for GD&S implementation entails considering the life cycle and staffing costs.

c. The costs that must be considered include:

- Initial software cost for software licenses.
- Initial hardware cost.
- Modifications to existing hardware that may be caused by the introduction of new hardware.
- Modifications to existing software that may be necessary to work in the new environment.
- Network implementation or modifications.
- Furniture and site modifications.
- Hardware installation.
- Software installation.
- Integration services.
- System test.
- Training of staff in system and new procedures.
- Software test.
- Data validation.
- Supplies such as media for data and output.
- Data collection and conversion of old data sets to new formats.
- Data maintenance.
- Supplemental utility programs.
- Maintenance contracts on hardware and software.
- Future system upgrades.

There are also unique costs brought about by the requirement for a National Geospatial Data Clearinghouse. These Clearinghouse costs include:

- Internet connectivity and system administration.
- Server hardware purchase and maintenance.
- Metadata creation for new and existing data sets.
- Maintenance of inactive data sets.
- Metadata distribution.
- Responding to data requests from Clearinghouse users.

Once the implementation is underway the management of expectations will begin. Here the goal is to deliver some operational capability through a pilot project which can be used to demonstrate capabilities using an initial geospatial data base. This serves to show the capabilities in an USACE environment while delivering on a functional requirement and to achieve a first success which should solidify management support. At the same time it allows the beginning of staff training and provides a set of lessons learned for the larger implementations to follow.

3-12. Staffing GD&S Positions

USACE Commands will not create new positions to support the requirements of ER 1110-1-8156. However, as GD&S

technology advances within the organization and becomes an integral part of conducting the mission, GD&S skills will become part of many job descriptions. This section provides some guidance on GD&S staffing.

Currently there are no formal GD&S titles in the Federal Civil Service; however, it is not uncommon to include specific GD&S skills in position descriptions or even to use informal titles, such as GIS Specialist, in job announcements. There have been attempts to develop them in the past with no success and there is unlikely to be success in the current environment which emphasizes general categories to promote staffing flexibility. A logical set of titles, grades and responsibilities is listed in Table 3-1. Representative paragraphs that may be adapted and inserted into job descriptions for GD&S-related positions are provided in sections 10.a-10.c of this manual.

a. GD&S Supervisory Position Skills. Oversees the development and use of the GDS to support the planning, design, operation and maintenance efforts of the USACE Command. Based on professional knowledge of the needs of the Command and an in-depth knowledge of the use and application of the GDS, advises and assists the Command management and GD&S committees, engineering and associated staff in the Command as to the uses of the GDS in the various Command activities.

(1) Monitors GDS function to determine problems and solutions. Keeps up-to-date on software and hardware availability and changes. Applies in-depth knowledge of the hardware and software to determine specific uses of the system and to establish procedures and policies for usage. Examines new applications software and hardware developments and assesses their use and cost effectiveness; initiates actions to purchase, when appropriate. Analyzes work for which software does not exist or where programs do not meet local needs, and determines feasibility of preparing local applications.

(2) Establishes procedures for use of equipment and software and updates as changes occur or as needed to correct problems. Recommends specific actions, policies, and procedures for supervisory/management review, approval and implementation. Maintains liaison with Command Systems Manager and advises manager on technical matters for contract actions involving GDS applications or requirements. Reviews comments on GDS applications and recommends changes in Command GDS policy.

(3) Recommends, coordinates, and schedules GDS training for Command employees. Provides specialized in-house training to GDS users. Maintains status report, software documentation, and GDS supplies.

(4) Participates in internal technical planning meetings and meetings with representatives from other USACE Commands for purposes of technical coordination. Participates in negotiation with universities and other research organizations for performance of contract work, in review of contract work, and in integration of services obtained.

b. GD&S Analyst Skills. Is responsible for planning and executing studies relating to the characterization of physical and cultural attributes of environments for use in USACE civil works projects and military activities/operations. Duties and responsibilities require a knowledge of and experience with GDS, computers, the geographic sciences, and digital geospatial data processing. Must be able to design and build new GDS applications using commercial software tools. Provides expert knowledge to other engineers and scientists (e.g., geologists, geographers, hydrologists, mathematicians, ecologists, and physicists) in setting up and conducting programs and projects and in the presentation of technical data. Formulates conclusions from spatial analyses to supplement that of the lead scientist.

(1) Plans and directs field studies to collect data to determine the quantitative relation between various environmental factors and components of structural and nonstructural alternatives for civil works and military projects. These studies include on-site data acquisition, airborne remote sensing missions, use of conventional surveying techniques, and use of automatic sensing and recording instrumentation.

(2) Participates in the direction of office studies, negotiates with other offices (USACE Districts and Divisions, U.S. Department of Agriculture, USGS, etc.) and organizations such as universities, research institutions, and commercial concerns, for existing information or cooperative work relying on own professional skills to review, interpret, and analyze information; formulate approaches; reach conclusions; and make recommendations. Develops methods and performs studies involving the comparison of geographical regions and specific sites for the purpose of determining degrees of analogy. Designs and develops computer programs to reformat acquired data as necessary for import into host GDS. In support of these and other studies develops and applies computerized methods for best portrayal of environmental data.

(3) Performs administrative duties appropriate for the technical work described, directs the work of professional and nonprofessional employees as assigned for accomplishing the work of assigned projects. Makes assignments and instructs employees of lower grade and checks performance for quality of work and rate of performance; is responsible for knowledge and observance of all regulations applicable to the work described.

Table 3-1
Sample GD&S Responsibilities

Title	Grade	Representative Responsibilities
GD&S Manager	GS 11-12-13	GD&S POC. Coordinates the GD&S efforts within the USACE Command. Serves as technical advisor to the Oversight Committee. Permanent member of the Technical Committee.
Geospatial Data Manager	GS 7-9-11-12	Maintains metadata and data for the Clearinghouse. Responsible for establishing and maintaining a geospatial data archive and library. Designs and constructs new databases. Provides recommendations on database issues to Technical Committee and GD&S Manager.
GD&S Spatial Analyst	GS 7-9-11-12	Uses GD&S technology to design and produce decision support products. Customizes and uses GDS tools to analyze spatial data of various types. Formulates conclusions from this analysis and recommends additional analyses to help further refine solutions.
GD&S System Operator	GS 7-9-11	Uses GD&S technology to produce decision support products. Knowledgeable of capabilities and operation of GDS. Constructs new databases. Customizes commercial GDS to perform specific applications. Oversees activities of GD&S Technician.
GD&S Technician	GS 5-7-9	Knowledgeable in the operation of GDS products. Operates peripheral devices to build and maintain geospatial databases.

c. GD&S Operator Skills. Is responsible for assisting in the planning and executing studies relating to the characterization of physical and cultural attributes of environments for use in USACE civil works projects and military activities/operations. Duties and responsibilities require a knowledge of and experience with GDS, computers, and digital data processing to acquire, store, retrieve, and display data in a form best suited to the user's needs. Must be able to design and build new GDS applications using commercial and custom software tools. Works cooperatively with other engineers and scientists (e.g., geologists, geographers, hydrologists, mathematicians, ecologists, and physicists) in setting up and conducting programs and projects.

(1) Plans field studies to collect data to determine the quantitative relation between various environmental factors and components of structural and nonstructural alternatives for civil works and military projects. These studies include on-site data acquisition, airborne remote sensing missions, use of conventional surveying/ techniques, and use of automatic sensing and recording instrumentation.

(2) Participates in the direction of office studies, negotiates with other offices (USACE Districts and Divisions, Natural Resources Conservation Service, U.S. Department of Agriculture, U.S. Geological Survey, etc.) and organizations such as universities, research institutions, and commercial concerns, for existing information or cooperative work relying on own professional skills to review, interpret, and analyze information; formulate approaches; reach conclusions; and make recommendations. Develops methods and performs studies involving the comparison of geographical regions and specific sites for the purpose of determining degrees of analogy. Designs and develops computer programs to reformat acquired data as necessary for import into host GDS. In support of these and other studies develops and applies computerized methods for best portrayal of environmental data.

(3) Performs administrative duties appropriate for the technical work described, directs the work of professional and nonprofessional employees as assigned for accomplishing the work of assigned projects. Makes assignments and instructs employees of lower grade and checks performance for quality of work and rate of performance; is responsible for knowledge and observance of all regulations applicable to the work described.

d. GD&S Technician Skills. Is responsible for assisting in the conduct of studies relating to the characterization of physical and cultural attributes of environments for use in USACE civil works projects and military activities/operations. Duties and responsibilities require a knowledge of and experience with GDS, computers, and digital data processing to acquire, store, retrieve, and display data in a form best suited to the user's needs. Works cooperatively with other engineers and scientists (e.g., geologists, geographers, hydrologists, mathematicians, ecologists, and physicists) in conducting programs and projects and in the interpretation, analysis and presentation of technical data.

(1) Participates in GD&S studies within the USACE Command. Assists in the interpretation and analysis of information; helps to formulate approaches and reach

conclusions; and makes recommendations. Helps to develop methods for performing studies involving the comparison of geographical regions and specific sites for the purpose of determining degrees of analogy. Writes computer programs to reformat acquired data as necessary for import into host GDS. In support of these and other studies, applies computerized methods for best portrayal of geographic data.

(2) Operates a GDS. Builds geospatial databases to specification by using a GDS application to enter coordinates and attribute data, operating a scanner, or other computerized method. Recommends improvements to the data entry system.

3-13. Training GD&S Staff

Required training in the GD&S area of expertise has not yet been formalized. The field is still evolving and academic studies have shown that curricula vary greatly. In general, the requirement is training in a field with a spatial component (e.g., geography, cartography, remote sensing, civil engineering, biology, oceanography, urban and regional planning, agronomy, forestry, landscape architecture, or geology) and course work in GD&S topics. In the present environment the progression is like the apprentice, journeyman, master sequence in the crafts.

3-14. Training Sources

There are a number of sources for GD&S training which make entry into the area and continuing education readily available.

a. USACE Training. The Training Symposium on Surveying, Mapping, Remote Sensing and Geographic Information Systems is sponsored by Headquarters USACE to transfer new technology developments to USACE users. This symposium is held every three years and provides short courses, plenary sessions and technical sessions. Exhibits of commercial and USACE capabilities are provided. Announcement of the symposium is made by a memorandum from HQUSACE (CECW-EP).

The Proponent Sponsored Engineer Corps Training (PROSPECT) Program courses are developed to meet unique USACE training needs. They are taught by USACE employees or by contractors and some provide continuing education credits. Of particular interest is a course entitled "GIS Introduction." The POC for PROSPECT courses is:

Commander
U.S. Army Engineering Support Center, Huntsville
ATTN: CEHNC-TD-RG (Registrar)
P.O. Box 1600
Huntsville, AL 35807-4301

GRASS Conferences and Courses provide instruction on the operation of the GRASS GIS developed by the Construction Engineering Research Laboratory. The POC for GRASS Conferences and Courses is:

Center for Remote Sensing and Spatial Analysis
P.O. Box 231
College Farm Road
Cook College
Rutgers University
New Brunswick, New Jersey 08903
coneil@ocean.rutgers.edu

b. Other DoD Training. The Defense Mapping School at Fort Belvoir, Virginia has several courses related to GD&S technologies including database production, remotely sensed imagery, GIS, cartography, and an introductory ARC/INFO course under development. The POC for the Defense Mapping School is:

Director
Defense Mapping School
ATTN: HR-DMSO
Fort Belvoir, VA 22060-5828

Training is also available through the Tri-Service CADD/GIS Technology Center and the CAD2 Contract. The POC is:

Tri-Service CADD-GIS Technology Center
U.S. Army Engineer Waterways Experiment Station
ATTN: CEWES-ID-C
Vicksburg, MS 39180-6199
601-634-2945

c. Universities. Hundreds of universities are now offering GDS programs usually integrated with a well established academic department such as geography, geology, forestry, civil engineering, or agronomy. Many universities also offer GD&S short courses. A listing is available in *Directory of Academic Departments Offering GIS Courses* published by The American Society for Photogrammetry and Remote Sensing (ASPRS) and the American Congress on Surveying and Mapping (ACSM).

d. Vendors. Vendors provide training in the operation of hardware and software as opposed to universities that emphasize concepts and applications to problems. This training may be acquired as part of a GDS procurement or through user groups and workshops.

e. Professional Meetings, Conferences and Symposia. Many professional organizations conduct technical meetings and offer workshops and training in GD&S technology arena. A few are listed below:

The American Congress on Surveying and Mapping (ACSM), Suite 100, 5410 Grosvenor Lane, Bethesda, MD 20814-2122

The American Society for Photogrammetry and Remote Sensing (ASPRS), Suite 210, 5410 Grosvenor Lane, Bethesda, MD 20814-2160

Automated Mapping/Facilities Management International (AM/FM), 14456 East Evans Avenue, Aurora, CO 80014-1409

The Association of American Geographers (AAG), 1710 Sixteenth Street, NW, Washington, DC 20009-3198

The Urban and Regional Information Systems Association (URISA), Suite 304, 900 Second Street, NE, Washington, DC 20002

The five professional organizations listed above jointly sponsor an annual meeting known as GIS/LIS. This large national meeting is designed to be an interdisciplinary educational and scientific meeting which promotes interaction among individuals and groups interested in the design and use of GIS, LIS, and related specialties and technologies. GIS/LIS workshops are taught at introductory and advanced levels and address a range of technical and managerial topics. Exhibits provide a chance to interact with vendors and learn about product enhancements and revisions and make near-direct comparisons among different products and thus save considerable time and effort if you are shopping for software, hardware, or services.

There are also Federal Government and State Government conferences which are dedicated to GIS such as the Federal Geographic Technology Conference, Exposition, and DataMart, the National Geodata Policy Forum, and the National GIS Council (NSGIC) Annual Meeting

Chapter 4
Requirements Analysis

4-1. General

a. Geospatial Data Systems are successful only when they comprehensively and consistently meet the needs of users. As such, the procurement, installation, and use of a successful GDS depends on well-defined user requirements obtained through a development stage known as requirements analysis. Requirements analysis is a detailed study of the needs of potential users of the GDS which helps cut down the costs of acquiring and maintaining a GDS. The requirements obtained during this study may come from a variety of sources including system operators, domain experts (someone with an in-depth knowledge of the functional area), requirements and software engineers, managers, existing systems and standards, specifications, and any other potential users. When completed, the requirements analysis should result in a clear statement of end-product characteristics, required production rates, estimated data volumes, and a cost/benefit rationale.

b. The documentation required to justify and validate the acquisition of Automated Information System (AIS) hardware and/or software is currently in transition between the General Services Administration (GSA) and the Office of Management and Budget (OMB). It is anticipated that policy and direction down to MACOM level will not be received until the Fall, 1996. Project Managers should contact their local Director or Chief of Information Management for guidance and assistance in the required documentation. Experience has shown that a more rigorous requirements definition is needed for GDS.

4-2. What is a Systems Requirements Analysis?

a. This section defines a systems requirements analysis and explains many of the elements of a good requirements analysis. GD&S require data to operate, so a separate, but similar, data requirements analysis is needed to ensure that the system has the data sets needed to perform the specified functions. The data requirements are discussed in detail in Chapters 7 and 8 of this manual.

b. The earliest stages of the system development cycle should involve information gathering about the system to be procured. A committee determines the GD&S needs of each potential user through written, verbal and face-to-face contact. These needs or requirements can include anything from types of data the GDS can handle to necessary product output types. Once the requirements have been gathered and compiled into a manageable set, they form the basis for a System Requirements Specification (SRS) which is used to determine all the external features of the GDS. Requirements analysis is the activity of understanding the application domain, the specific problems and needs of the users, and the constraints upon possible solutions. When a requirements analysis is done properly, it is possible to match the description contained in the SRS to several systems and pick the GDS which best suits all the users in the agency.

c. One of the misconceptions about requirements analysis is the "what versus how" dilemma. A requirements analysis study should only include any external behavior of the GDS, however, many people tend to confuse the external and internal behaviors and try to describe not only their needs but also how the system should solve those needs. The "how" aspect of the system should be left to the design phase of the development cycle which comes after the requirements analysis has been completed. GDS system requirements are capabilities needed by users to solve GDS problems (e.g., performing a site suitability analysis which involves graphical and textual data and multiple maps with differing projections), and these requirements must be met by a system component (e.g., computer system, software, data, etc.) in order to satisfy a contract, standard, or system specification. System requirements can be broken down into seven main categories:

- Functional requirements specify transformations that take place in the software system (i.e., the inputs and outputs for each function).

Example: The software shall identify all overshoots and undershoots by placing a box around the dangling end of each line segment.

- Behavioral requirements specify the way the system reacts to external and internal stimuli (i.e., what causes certain activities to start, stop, etc.).

Example: The program shall pause and return to the previous condition when the *escape* key on the keyboard has been depressed.

- Performance requirements specify timing, throughput, and capacity of the system.

Example: The system shall be able to return the geographic location of any address in under 3 seconds.

- Operational requirements specify how the system will run and communicate with its human users.

Example: The system shall provide a help facility, accessible from software, that describes the system components.

- Interface requirements specify characteristics of the human/computer interaction, such as screen layouts and validation of user data entry. In the case of system, an interface requirement can also specify hardware interfaces.

Example 1: Error messages must be displayed on the screen starting at row 1, character position 1.

Example 2: The system must be targeted to run on Sun workstations with the Solaris 2.1 Unix operating system.

- Quality Requirements specify system reliability, useability, and maintainability.

Example: The system must not fail more than 3 times during 1,000 CPU hours of operation.

- Constraints specify restrictions that impact the system in some way.

Example: Each line segment will be limited to 255 coordinate pairs.

d. In addition, there can also be security requirements when the system is going to be working with classified data and data requirements dealing with the validation and accuracy of the data sets passed to the system.

e. When the task of gathering the system requirements is over, it is important to sort through them to build a manageable set for the SRS. You will probably find that many requirements have been repeated several times although they may be worded differently. Others may be ambiguous, needing additional input from the source of the requirement. The Carnegie Mellon Software Engineering Institute has developed guidelines for generating such manageable sets. In these guidelines, they describe the seven requirements error types to watch out for when organizing the requirements set:

- System requirements are ambiguous if there is more than one interpretation. Ambiguity arises due to the pitfalls of natural language.

Example: Total item count is taken from the last record.

Problem: How do you interpret last record?

- System requirements are incorrect if some fact within the requirement has been misrepresented.

Example: For all computations, use the northwest corner as the origin of the data set.

Problem: What if the southwest corner were the origin for some of the data sets?

- System requirements are incomplete if one or more necessary facts (or requirements) have been omitted.

Example: Error messages must be displayed on row 24 of the terminal screen.

Problem: How will the system display multiple, simultaneous error messages?

- System requirements are inconsistent if they are in conflict.

Example: C must be computed as A + B
C must be computed as A - B

Problem: C has two definitions.

- System requirements are volatile if they are susceptible to change.

Example: The system shall handle only one data set at a time.

Problem: Although this may satisfy current needs, what happens if you need to work with multiple data sets simultaneously?

- System requirements are untestable if no cost-effective way exists to verify them.

Example: The system shall have a good user interface?

Problem: There is no way to define a "good" interface, so there is no way to verify it.

- System requirements are nonapplicable if they are not relevant to the problem.

Example: The operator of the system must be standing upright during operation of the system.

Problem: This requirement has nothing to do with the development or procurement of the system.

f. These guidelines should help throughout the systems requirements analysis process. The requirements analysis is described from start to finish in Chapter 4. Remember, that once you have formulated an SRS from your requirements

set, it should be used as a "bible" from which a GDS will be procured. The SRS can help identify potential errors before they become costly to fix, so you should treat this stage in the GDS life cycle with the utmost importance.

4-3. Why is Requirements Analysis Important?

A requirements analysis can appear to be an overwhelming task to complete for a Command-wide GD&S procurement. However, the benefits of performing a comprehensive analysis prior to system acquisition are well-documented and easily justify the effort.

Errors are much easier and less costly to correct when they are detected early in the development stage rather than later on. The reason for this is that one component in a system will often build upon the capabilities of another until you have a fully-integrated system. In any failed system development, it can be demonstrated that when one component is missing certain required capabilities, this problem will propagate itself through the rest of the system. Other components will not be able to meet certain needs, and so on with a cumulative effect on the entire system. This lesson is equally true for an "off-the-shelf" GDS and a custom-developed GDS.

The document most often used to record the system requirements is the System Requirements Specification (SRS). It clearly explains every detail of the system without ambiguity or error, so that the buyer can make intelligent choices about the system to be procured. In addition, this document also serves as contract between the buyer and developer/supplier; the GDS is not complete until the developer/supplier has met every requirement contained in the SRS.

4-4. Who Should Perform the Requirements Analysis?

In "A Process for Evaluating Geographic Information Systems," Guptill writes:

The first problem faced by any organization considering the implementation of a [GD&S] is to determine who should perform the requirements analysis. Requirements Analysis' for successful [GD&S] installations have been performed by in-house staff, contractor staff, or through a combination of both approaches. There are many valid reasons for opting for any given approach, however, the desired result is the same, to develop a comprehensive assessment of the analytical capabilities and products required by potential [GD&S] users. The requirements of the users can then be matched with system capability to determine optimal configurations for the organization's [GD&S] procurement.

In-house staff inherently have a greater understanding of the tasks which are to be considered for automation through [GD&S] technology. This unique knowledge may justify training staff members in [GD&S] technology and requirements analysis techniques so that the requirements analysis may be performed in-house. In cases where existing staff members have expertise in GD&S's there may be little reason to consider bringing in outside assistance.

When staff time or skills are not available, or when new programs and concepts are being proposed that staff is not experienced with, outside resources may be required to perform the requirements analysis. Assistance may also be available from resources within the parent agency of the organization considering the [GD&S] implementation. Assistance in developing Requests for Proposals (RFPs) for requirements analysis services may similarly be available within the agency.

The important element in determining who should perform the requirements analysis is assuring that the provider of the service has a thorough understanding of both [GD&S] technology and the operations of the organization. When an outside organization is brought in to perform the requirements analysis, it is the responsibility of the technical representative or point of contact for the requirements analysis services procurement to assure that the contractor fully understands the organization's products, services, missions, and needs.

Possible conflicts of interest should also be considered before a final determination is made as to who should perform the requirements analysis. Organizations and individuals may, in some instances, have a vested interest in certain hardware or software types, and may be inclined, whether intentionally or unintentionally, to bias the results of the requirements analysis toward particular systems. The objective of the requirements analysis is to identify the needs of an organization and then to select the [GD&S] that best fits those needs, if such a system exists. All reasonable effort must be made to assure this goal is realized, including assessing possible conflict of interests, or biases, on the part of persons or organizations that potentially could perform the requirements analysis.

4-5. Performing a Requirements Analysis

The purpose of this section is to lay out a generic framework for performing a requirements analysis with some key tips where appropriate.

a. Elicitation. The first step in performing a requirements analysis is to obtain system requirements from all persons involved with the use of the GD&S system. This

step, known as requirements elicitation, can involve any of the following activities:

- Interviewing.

  - Face-to-face contact with potential users, manager, etc.

  - Should identify relevant positions from a formal organizational chart.

  - Identify the work flow interactions between users and the rest of the organization.

  - Ask context free questions such as:

    "Who else should I talk to?"

    "Who else may use the system?"

    "Who else interacts with you?"

  - Inform interview candidates ahead of time and give them any relevant material.

  - Secure an adequate time commitment from candidates.

  - Give the person reasonable courtesies in terms of answering your questions.

  - Periodically confirm your understanding of the person's responses.

  - Summarize your understanding at the conclusion of the interview.

- Brainstorming.

  - A simple technique for generating ideas.

  - Can be used for generating alternate viewpoints of the problem.

  - Works best with groups of 4-10 people.

  - Outcome depends on the expertise and knowledge base of the participants.

  - Generation phase: a leader provides a seed expression to the problem.

  - Generation phase: participants freely generate ideas relevant to the problem.

  - Generation phase: all ideas are placed on a large board or sheets of paper.

  - Generation phase: should be stopped when ideas become low (~ 15-20 min.).

  - Consolidation phase: ideas evaluated, outliers removed, related ideas combined.

  - Consolidation phase: remaining ideas are grouped and classified.

- Scenario generation.

  - Determine needs through real world system usage scenarios.

- Rapid prototyping.

  - Quick user interface or basic system shell construction.

- Modeling.

  - System portrayal through means such as data flow diagrams.

b. Analysis. The next step in the process is to organize the information received from the elicitation process. At this stage you want to remove any requirements errors you can find by looking for ambiguities, inconsistencies, and any of the other seven requirements errors previously mentioned. You also want to remove any requirements which are duplicates. The main goal here is to build an organized set of requirements that clearly state the system and its behavior with as few requirements as possible. If additional information is required, you should go back to the elicitation stage before continuing, because you will need the final requirements set before determining feasibility.

c. Feasibility. It may or may not be possible to procure a system with all the capabilities requested by users, so you must next perform a feasibility analysis. During this phase, it is a good idea to conceptually plan the system without getting into too much detailed design. You should also speak to representatives of commercial GDS vendors to discuss your requirements and how their systems could be used to meet them. System modeling through data flow diagrams, decision tables and trees, and control flow diagrams is a good way to visualize the system from the given requirements set. Matching the capabilities of possible GDS solutions against these models will give you an understanding of what will be required to build the necessary GDS, either as a custom software development or as a customization of a commercial GDS. If specific

requirements are making the GDS too expensive or risky to build, you must decide whether or not to write the SRS and have the system developed as-is or to eliminate the infeasible requirements.

d. Specification. Without an SRS, the contractor/in-house development team would have no idea of what was to be built, your users would be left with false expectations of the GDS, and there would be no way to test if the solution meets all the needs captured during the elicitation and analysis phases. Therefore, you must document the external behavior and qualities of the system while excluding any internal information concerning how the software and hardware operates and the algorithms it uses. The SRS has four roles in the development life cycle: (1) it is the primary input to the design team; (2) it is the primary input to the system test planners; (3) it controls the evolution of the system; and (4) it communicates an understanding of the system requirements. The SRS should be understandable to anyone who reads it regardless of their background, and should contain the following qualities:

- An SRS is correct if every requirement stated therein helps to satisfy a user need.
- An SRS is complete if every user need is satisfied by a system that satisfies every requirement in the SRS.
- An SRS is unambiguous if every requirement stated therein has only one possible interpretation.
- An SRS is consistent if no subsets of the requirements stated therein conflict.
- An SRS is logically closed if it specifies a response for every conceivable stimulus in every conceivable state.
- An SRS is organized if its readers can easily locate information.
- An SRS is modifiable if it can be easily changed when errors are found, or when it needs to be changed due to changes in requirements.
- An SRS is traced if the origin of each of its requirements is clear.
- An SRS is traceable if other documents can reference its requirements easily.
- An SRS is concise if it cannot be made shorter without jeopardizing other qualities of the SRS.
- An SRS is verifiable if there exists a finite, cost effective technique to check that each requirement in the SRS is satisfied by a system.
- An SRS is annotated by importance if the relative importance of each requirement is indicated.

4-6. Performing a Data Requirements Analysis

For most GD&S, the data are the largest cost component, often eclipsing the cost of the hardware and software combined. It is critical that they be properly managed from initial design to long-term archive. The data requirements analysis is the first step in building or acquiring a geospatial database. It is used to both define the content and format of the data as well as to inform potential users of the intent to acquire/produce the database. It is conducted similarly to the system requirements analysis, with the objective of establishing the minimum set of requirements that will meet the near- and long-term needs of the users. It should establish the database:

- Structure.
- Content.
- Scale/resolution.
- Accuracy.
- Currency.
- Any special user requirements.

a. Structure. The structure of the database defines its basic makeup. The two most common geospatial database structures are vector, such as road networks or engineering drawings, and raster, such as gridded elevation matrices or digital imagery.

The database structure is established by the application(s) of the database and the systems that will be used. Applications that involve detailed analysis of lineated features, such as road or stream networks, require attributed vector databases. Raster databases are ideally suited to applications that involve the boolean combination of areal, layered information. The computation of cross-country mobility, which is a function of ground slope, surface roughness, vegetation coverage, and other terrain characteristics, is an example of a query that is often performed using raster databases. The structure of the database should be determined very early in the requirements analysis.

b. Content. The content of the database is determined by the applications for which it will be used. If the database

has a vector structure, then it is necessary to determine the specific inclusion criteria for features and attributes. For example, if roads are required, to what level (highway versus dirt cart path), and how richly must they be attributed (e.g., surface material, number of lanes, whether the road has a median, whether the road access is controlled, whether the road is one-way or two-way, etc.)? Collecting unnecessary features and attributes adds to the cost of the database, so it is important to establish the minimum set of features and attributes that satisfy all user requirements.

If the database has a raster structure, then the thematic layers must be determined. Raster databases may have only one layer, such as a digital image, or it may have many layers, such as those needed for the cross-country example discussed above. As with vector databases, increasing the information carried in a raster database increases the cost of production, distribution, and archive, so it is important to establish the user requirements early in the design.

c. Scale/Resolution. Scale refers to the collection scale of vector databases. A collection scale of 1:24,000, for example, implies that the vector database has a feature content that is roughly equivalent to a hardcopy USGS 7.5-minute Quadrangle. Resolution refers to the size of the pixels in the layer(s). Resolution may be expressed as a distance on the ground (e.g., 10 meters) or as is common for raster databases produced by scanning hardcopy products, pixels per inch (e.g., 300 dpi). The scale/resolution has a significant effect on the ability to perform certain types of queries; a database with insufficient scale/resolution may preclude analysis at the level of detail required, but a database with an excessive scale/resolution is more expensive to build and has a negative impact on storage volume and processing times.

d. Accuracy. The accuracy of a database refers to several factors, including coordinate accuracy, attribute accuracy, logical consistency, and completeness.

(1) Coordinate accuracy. Coordinate accuracy refers to the accuracy, expressed as a distance and a confidence factor, of the geographic positions in a database. The coordinate accuracy of a database is influenced by, but not controlled by, the scale/resolution.

(2) Attribute accuracy. The attribute accuracy reflects the confidence in the codes and attribute values assigned to features in a vector database.

(3) Logical consistency. Logical consistency is a measure of the relative positioning of features in a vector database. An example of a break in logical consistency is a building captured on the wrong side of a road. While both the road and the building may be within their allowed coordinate accuracies, their relative positioning is not consistent. Logical consistency is most often expressed as the percentage of features that are consistent with all surrounding features. SDTS expands this definition to include the general fidelity of the data capture, considering such factors as the presence of overshoots and undershoots, topological integrity, and graphic presentation.

(4) Completeness. Completeness refers to the percentage of features in the real world that are captured in the database, within the capture rules of the particular database. Completeness, which deteriorates over time as new features are added to the real world, is measured at the time of database construction.

e. Currency. The currency of the database refers to the elapsed time since collection of the source material. Areas change at different rates, e.g., cultural features in suburban areas change much more rapidly than in sparsely populated regions, so currency requirements for databases often vary according to the area being captured.

f. Special user requirements. This section of the requirements analysis captures any unique needs for the database. These might include: on-line access to the database by the users, unique archive requirements, or special services such as on-demand datum or coordinate transformations.

Chapter 5
Implementation Plan

5-1. Introduction

a. It is a requirement that all USACE Commands develop and maintain an individual multi-year GD&S Implementation Plan (IP) for purchases of dedicated GD&S hardware and software systems. Some USACE Commands have developed and maintain implementation plans. One example is available on the Internet at Universal Resource Locator (URL) http://corps_geo1.usace.army.mil. Some of the considerations used by the Commands in developing their IP's includes: resolution, format and use of presently encoded data; anticipated needs for future data and a GD&S model; compatibility of systems division-wide; Information Management support needed; coordination of GD&S needs and uses across disciplines (e.g., Real Estate, Engineering, Planning and Construction, Operations); and costs.

b. Development and execution of a GD&S Implementation Plan is necessary in order to guarantee the successful and effective installation of a GD&S. In order to reduce risks and minimize excessive costs and redundancy, a comprehensive design and timeline must be created in order to reduce the likelihood of failure. A three to five year timeline driven by specific performance milestones and pilot projects assures progress and manageability. Even if the IP is not acted upon, the document will serve as an informative report on user needs and may benefit other agencies interested in implementing a GD&S.

c. Figure 5-1, taken from an Environmental Protection Agency (EPA) system design (EPA 1993), illustrates the time and resource savings that can be achieved if a system has been properly engineered, such as with an implementation plan. Long-term maintenance costs will be reduced and system implementation will require fewer resources and cost less. The development of an IP will increase costs in the short run, but during a three to five year life cycle, the cost savings during implementation and maintenance will be significant.

5-2. GD&S Implementation Plan Contents

The following sections describe the required content and recommended format of a GD&S Implementation Plan (IP). USACE commands may reformat the IP at their discretion but must include the required information.

a. Background. The background section of the IP summarizes the objectives of the GD&S Implementation Plan and explicitly states the need to execute this IP consistently and comprehensively. This section should also provide examples of the positive impacts of implementing this IP.

b. Scope.

(1) General.

This section of the IP provides some general information of GD&S technology and its application. The following is adapted from existing USACE GIS Implementation Plans:

GD&S is a powerful tool for combining a variety of data types into a common format. The system will speed any analyses that use spatially referenced data by automating processes that were often done by hand. Due to improved technology more information can be managed and manipulated. The sophistication of the analyses that now can be performed is enhanced over what can be achieved using only printed maps. Immediate updating of map files, viewing of multiple classes of information, and iterative "what if" analyses are a small sampling of the available capabilities.

All disciplines in the District must work cooperatively to share geospatial data whenever possible. Increased awareness of data availability and greater sharing of data between disciplines reduces the potential for loss of valuable data (e.g., "old" aerial photos or maps) and reduces costly replication of data collection. The most cost- effective approach to the development and use of a District-wide geospatial data inventory involves consideration of uses for each geospatial data set by the community of potential users, avoiding a limited purview engendered by looking at single applications. Data created by one corporate entity should be available for sharing with all spatial data users within the District or by District customers. Use should not be restricted or precluded by file format, operating system, object description incompatibilities, and so forth.

(2) Existing uses.

This section of the IP will list and describe current GD&S systems implemented and will discuss their individual strengths and weaknesses. Each GD&S description will include whether the data is raster or vector based, what the data sources are, what analyses are exercised, and what modeling is done. In addition, report on the current status of the GD&S in regard to its implementation plan and whether the GD&S has been effective.

(3) Future uses.

This section of the IP provides examples of current projects that could profit from the technologies of a GD&S. Include projects that handle large amounts of data that could be

analyzed more effectively in a geographical environment. Describe the needs or potential needs of the project and then show how GD&S technology could simplify and enhance the capabilities of the project.

(4) Needs assessment.

The needs assessment section of the IP lists and prioritizes the requirements presented in Sections 5-2.b (2) and (3) and describes what GD&S solutions would meet the requirements. Assess the requirements and describe the potential benefits of implementing a GD&S in these environments. Multiple sites can find great benefit in sharing data across networks and therefore reducing overall cost.

(5) Communication.

This section of the IP discusses how the GD&S technology and capabilities will be brought to the field through communication. Modes of communication will be compared based on availability and ease of use and may include newsletters, electronic mail, etc.

(6) Awareness of technology.

This section of the IP discusses the importance of the awareness and education of decision-makers of the technology and benefits of GD&S. Awareness may be spread through the use of video tapes, demonstrations and seminars.

c. Design guidance.

(1) Hardware.

The hardware section of the IP presents standards and requirements that restrict the selection of GD&S hardware. Describe the federal and industry-wide standards (GOSIP, POSIX, etc.) that may be relevant. Make sure the standards that are imposed on the GD&S hardware are necessary and appropriate in the light of standards being modified regularly. The major aspect of a GD&S system that must be standardized is the protocols and transports it uses to communicate with other systems or networks. Attention should be given to how well the hardware will fit with existing systems and how well the hardware can be maintained and upgraded. HQUSACE can provide numerous examples of GDS that became dependent on hardware platforms that were obsolete before the system was fully operational.

The decision to implement a multiple system (networked, redundant CPUs) solution or a single (stand-alone) system solution will be made based on the system's user load, throughput, and the requirements for reliability. Multiple system implementations allow more users to be online simultaneously and still perform complex spatial analyses. If a single CPU fails, the multiple system solution will allow continued operation with low to medium effect on processing times. Multiple systems can also effectively share hard disks for storing large databases. However, successfully maintaining multiple systems require additional labor for administration, which can be costly and will demand more strict system resource procedures.

The single system is a simple and cost-effective solution, but it has many drawbacks. On the plus side, the single system features lower maintenance requirements. However, the dependence on a single machine can be disastrous if a system failure corrupts data or delays time-critical projects. The single system limits the number of users that may use the GD&S drastically; in many cases only one user may effectively use the system at one time.

The specific hardware platform that is chosen will depend on the system requirements. A single-user single machine system should use a low-cost solution, such as a 586-based PC configured with at least 16MB RAM and a 500MB hard drive. This system may run using either DOS/Windows or OS/2 and will provide impressive performance for the price. However, if user requirements specify the need to access information from other Districts or from other sources on the Internet and require multiple users, then a UNIX based hardware platform is the best solution. A minimum configuration would be a Sparcstation 20 with 32MB RAM and a 1 GB hard drive. Additional peripherals should be added to each system, such as backup drives, CD-ROM drives, digitizers, scanners and printers.

(2) Software.

The software section of the IP presents standards and requirements that restrict the selection of GD&S software. Describe important capabilities that are required for the proposed GD&S. Issues involving raster and vector compatibility, interoperability with CADD systems, data exchange with cooperating agencies, and importing important nationwide databases should be addressed.

(3) Data formats.

The data format section presents standards and requirements that restrict the selection of GD&S data formats. Discuss data format conversion for importing and exporting data with cooperating agencies and potential users. Describe the different formats that the GD&S will be able to input and output.

(4) Connectivity.

This section of the IP presents standards and requirements that restrict the selection of GD&S networking protocols and transports. This may be done by standardizing on a particular operating system and networking protocol, such as Unix/NFS/Ethernet. This section will also mention the importance to be able to share data with important data sources or other systems that may not be GD&S. An important example would be the capability to connect a raster-based GIS to a vector-based CADD system.

d. Data management.

(1) Data access and exchange.

This section of the IP discusses how data will be accessed and exchanged within the GD&S and with outside organizations.

(2) Data exchange formats.

This section of the IP discusses what formats will be used for data exchange with an emphasis on compatibility and minimizing data loss. The implementation of the Spatial Data Transfer Standard (SDTS) will be addressed also.

(3) Data stewardship.

The data stewardship section of the IP discusses the importance of the centralized database stewardship and how that improves security and minimizes data misuse.

(4) Data maintenance.

This section of the IP discusses who will be responsible for data security, data backups and data updates for each database. Specific procedures and equipment for data backup shall be addressed.

(5) Data archiving.

In this section the IP will discuss the standard operating procedure for performing routine daily and weekly data and system backups.

(6) Data quality.

This section of the IP discusses how quality assurance and quality control will be maintained for each database. Data acquisition standards and procedures will be addressed. Data sources may provide data in non-standard formats which will require that a standard format will be defined along with a procedure to convert all incoming data to the standard format. The standard format for data exchange is the SDTS. It specifies exchange constructs, addressing formats, structure, and content for spatially-referenced vector and raster (including gridded) data.[1]

e. Scope of existing and proposed systems.

(1) Existing systems.

This section of the IP will give detailed system configurations of each currently existing GD&S or related system. The list of system hardware should include workstation name and type, memory installed (RAM), secondary storage devices, tape backup devices, input devices (mouse, tablet, digitizer, scanner), and printers. Describe the operating environment and the networking protocols be used. List all the GD&S related software installed on the computer, including databases.

(2) Pilot projects.

This section of the IP will discuss all pilot projects that have been done or are being planned. The projects will be used to generate a list of lessons learned and a list of additional requirements for the proposed GD&S. Describe what GD&S was used and evaluate the system for its effectiveness.

(3) Proposed systems.

This section of the IP will describe what the proposed system hardware and software configurations will be for the GD&S and how these decisions were made. Pilot Project evaluation is an important step in formulating a wise system configuration.

f. Life cycle costs and justification.

(1) Hardware costs.

This section of the IP will discuss the costs involved in acquiring the hardware for the proposed systems. Hardware costs include the cost of the central processing units, monitors, keyboards, input devices, digitizers, modems, network connections, secondary storage devices, tape backup systems, printers, scanners and other non-standard hardware peripherals.

(2) Software costs.

This section of the IP will discuss the costs involved in acquiring the software for the proposed systems. Software

[1] Paraphrased from the Spatial Data Transfer Standard (FIPS 173) Fact Sheet, July 1994.

costs include operating system support, GD&S software (i.e., Microstation, ERDAS, ARC/INFO, GRASS), database server software (i.e., ORACLE, Informix) and software maintenance packages.

(3) Data acquisition.

This section of the IP will discuss the acquisition of data for the proposed system. Sources and costs for each database will be listed as well as sources for optional data sets.

(4) Database development costs.

This section of the IP will describe how the GD&S database will be designed. Efforts should conclude with the identification of the data dictionary used.

(5) System benefits.

(a) Tangible benefits.

This section of the IP will discuss the benefits of using a GD&S over a manual method which may include the following issues: easier data entry, efficient data manipulation, credible data analysis, faster data output, improved access to current data, ability to perform complex analyses not possible with manual methods, and manpower savings.

(b) Intangible benefits.

This section of the IP will discuss the intangible benefits of using a GD&S which may include the following issues: extended use/reuse of mapping data, ability to analyze more alternatives, common framework for analysis and data sharing, credibility and repeatability of analysis, interdependence of organizations, modeling, and morale and prestige.

g. System implementation.

(1) Personnel requirements and costs.

This section of the IP describes the staffing requirements of the GD&S with emphasis on the source of staffing and the day-to-day activities. GD&S is a shared resource that should be manned and funded as a cooperative basis through all interested parties. Staffing can best be analyzed by reviewing the day-to-day activities that will be required (paragraph 3-10 provides some example position descriptions that may be needed).

(2) Training requirements and costs.

This section of the IP discusses the issues involved with acquiring or developing skilled personnel for the operation
(c) Document lessons learned.

and maintenance of the GD&S. The GD&S industry is growing both in number and in technology, so there will be a continuing need for advanced training. GD&S education can be pursued through contractor seminars/classes or through college course work. Having highly-skilled staff must be a high priority in order to maintain a complicated system such as a GD&S.

This section will discuss the training program that will be used to maintain a highly skilled and educated staff. Multiple disciplines must be supported to ensure that a properly prepared staff is available. This may include scheduling contractor based or college course base GD&S training since there is a lack of GD&S dedicated seminars.

(3) Acquisition strategy.

This section of the IP details the sources of funding and contract vehicles for the acquisition of the proposed systems. System acquisition may be fragmented over many years, so be sure to provide a complete timeline for all procurements. This section will discuss the issues involved in hardware and software acquisition after the systems arrive. Describe the ramp-up time required for staff to learn how to use the systems appropriately. An orderly ramp-up can be achieved through an active system administrator who maintains standard operating procedures for the system. In addition, describe who will be in charge of allocating GD&S resources to competing projects.

This section will also include tabular timelines detailing the time and cost of system acquisition, training, user group activities, database development and pilot projects. Milestones shall be established so that the timeline may be followed accurately and with performance evaluations. Provide sources of funding for all acquisitions. Create system configuration schematics. Describe the procedure by which ongoing evaluation of the implementation can be made with meetings at least annually.

(4) Time frame.

(a) Mission assignments.

This section of the IP can establish mission assignments with specific staffing requirements and goal-oriented projects.

(b) Milestones.

This section of the IP will establish short- and long- term milestones so that evaluations can be performed and progress can be reviewed regularly.

(c) Document lessons learned.

This section of the IP may describe how a separate "Lessons Learned" document will be created and updated throughout the term of the Implementation Plan. This document will provide useful insight into the implementation of a GD&S and offer advice for future revisions of the IP.

h. Evaluation plan.

(1) Geospatial data and systems technical committee.

This section of the IP assigns the responsibility of performing IP evaluations to the Geospatial Data Technical Committee.

(2) Time table.

This section of the IP describes when, where and how often specific evaluations will occur. The time table for these evaluations will conform to Paragraph 7 of Engineer Regulation 1110-1-8156.

i. Conclusions and recommendations.

This section of the IP discusses the reasons why a GD&S is needed. It should state that the IP presents the justifications, the means and a plan of action for the implementation of a GD&S. It should also state that as technology and the GD&S industry continues to advance, the plan must be reviewed annually and updated as necessary. This section will provide a list of recommendations that summarizes the solutions to the requirements elicited during the requirements analysis.

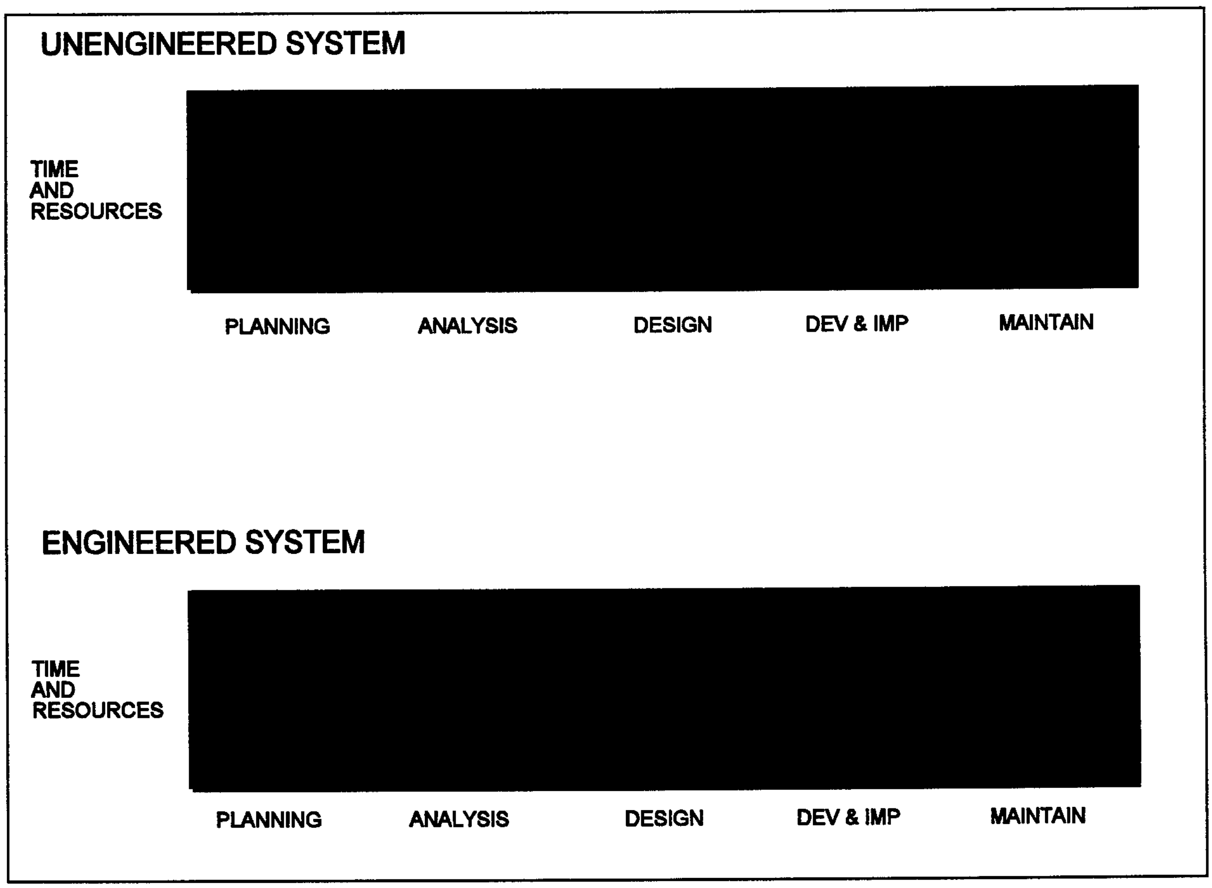

Figure 5-1. System development life cycle comparison

Chapter 6
GD&S Performance Evaluation Criteria

6-1. Introduction

The objective of developing performance evaluation criteria is to establish a protocol for assessing the periodic status of progress toward the goals set in the Implementation Plan by the USACE Command. The key aspect in evaluating the performance of a GD&S implementation is determining whether the system is satisfying user needs, both inside and outside USACE. Selden (1987) proposed a definition of GD&S success which provides an overall framework to construct specific performance evaluation criteria. This definition is as follows:

> *[GD&S] success is defined as the meeting of organizational requirements for the collection, management, analysis, display, and distribution of geographic/geographically-defined information commensurate with the level of investment over time. Furthermore, [GD&S] success is measured by the degree to which it becomes integrated into, and a part of, an organization's overall information resources infrastructure over the long term.*

6-2. Benefits of GD&S

The documented results of a GD&S implementation performance evaluation can have a beneficial impact in several critical areas including: the preparation of budget requests, the creation of information management plans and assisting Headquarters in responding to a variety of data requests.

6-3. Schedule of Evaluation

The evaluation must initially be performed one year after the approval of the Implementation Plan. This will provide a first impression of how the plan is being carried out. After this initial evaluation, re-evaluations of progress must be made on an annual basis. An information copy of evaluations is to be sent to HQUSACE (CECW-EP-S).

6-4. Issues Associated with the Development of Evaluation Criteria

The evaluation criteria listed in this section cover four major areas: technical, financial, organizational, and personnel. Technical evaluation criteria include software, hardware, data, interfaces, and standards. A good starting point in the development of criteria are the five basic objectives, outlined by the General Accounting Office (GAO, Evaluating the Acquisition and Operation of Information Systems, 1986) for the acquisition and operation of information systems:

a. Ensure system effectiveness. As stated in the implementation plan and requirements analysis, system effectiveness is measured by determining whether the system performs the intended functions and whether users get the information they need, in the right form, in a timely fashion. The following basic questions should be answered and explained with examples:

- Did you accomplish the goals of using the GD&S that you stated in your implementation plan/requirements analysis?
- Does the GD&S implementation produce real products which meet operational needs?
- Does the GD&S implementation provide meaningful support to senior decision-makers?
- Does the use of GD&S implementation promote inter-disciplinary, inter-departmental, intra-organizational, and inter-organizational coordination and cooperation?

To answer these questions, analysts should develop a table which lists the goals and requirements from the Implementation Plan in one column and GD&S implementations in the other. An example is provided at Table 6-1.

Table 6-1
Example Goals/Implementations

Goals/Requirements	GD&S Implementation
Develop a comprehensive inventory and tracking system for all spatial data holdings.	Using GD&S to convert all spatial data into a common format or projection and developing on-line system to display current data holdings.
Educate staff in the operation of an GD&S.	Setting up and conducting a GD&S training course; developed an GD&S; in-house GD&S training manual.

b. Promote system economy and efficiency. An economical and efficient GD&S implementation uses the minimum number of information resources to achieve the output level the system's users require. Under this category, a measure of GD&S implementation success would be the answers to the following questions:

- Can you describe instances where you saved time or money by using a GD&S, had fewer errors in products, were able to locate data to share?

- Are you a successful participant in the Clearinghouse for provision of data? Do you provide metadata to the Clearinghouse? Have you received data through the Clearinghouse?

To quantify the answer to these questions, a comparison could be made of the cost (money and time) of doing business before and after the implementation of GD&S. For example, another table could be developed which lists applications/projects and resources (time, money, labor) to complete. This assumes that adequate data exists on the costs associated with projects before GD&S was implemented, a form of information that should be gathered with the requirements analysis. An example of such a table is shown at Table 6-2.

c. Protect data integrity. Data integrity requires that systems have adequate controls over how data are entered, communicated, processed, stored and reported. Under this category, consider how the GD&S implementation has affected the organization and management of your data. Consider such items as:

- Data redundancy -- Have multiple copies of the same data set been eliminated?

- Clarity of presentation -- Is the data being presented in a more meaningful way?

- Time and costs associated with data entry -- Is data converted and stored in a common GD&S format?

- Data stewardship -- Has responsibility for maintenance of data and preparation of metadata been established?

- Data exchange formats -- Can data be easily transferred into and out of the system?

d. Safeguard information resources. Information resources, which include hardware, software, data, and people, need to be protected against waste, loss, unauthorized use and/or fraud. This category basically deals with data security and system access. Some of the implementation evaluation criteria in this section include:

- Are there standard practices and procedures in place to process sensitive or classified data within the GD&S?

- Is there a system configuration plan in place?

- Are proper file access privileges in place to control unauthorized access to system files, the operating system, common data directories, and user directories?

e. Comply with laws and regulations. Compliance with laws, regulations, policies, and procedures that govern the acquisition, development, operation, and maintenance of information systems must be ensured. A question like the following evaluates this:

- Has the implementation of a GD&S put your organization in a better position to comply with government regulations and reporting requirements not only with system operation and maintenance issues but also with respect to projects associated with litigation support which are related to government regulations?

f. Additional evaluation criteria. Other questions to consider in the implementation evaluation are:

- Are you growing in your use of GD&S technology, i.e., are you using it in new ways?

- Is the GD&S allowing you to do more work, new work, and has it increased your customer base?

- Are more people interested in the GD&S? Is your success attracting new customers?

Table 6-2
GD&S Effectiveness Evaluation

Application/Project Name: ______________________________

Project statistics	Before GD&S	After GD&S
Dollar cost		
Labor hours		
Time to complete		

Additional project statistics could be developed as necessary. To fill in the project statistics, break down the project into tasks and determine the time and cost associated with each task. For example, consider a project which requires the determination of chemical plume spreading in groundwater. A detailed table listing tasks and costs could be developed as follows:

Labor requirements with and without GD&S

Task/Product (Hours)	Technician Labor (Hours)	Engineer/Scientist ($)	Cost
	Before/After	Before/After	Before/After
Data Entry			
Well Logs			
Contour Maps			
Cross Sections			
Time-Series Plots			

Chapter 7
Geospatial Data Issues And Standards

7-1. General

The database itself is normally the most expensive component of a GDS and represents a valuable resource to the Command. The design, development and long term maintenance of a comprehensive geospatial database is a sizable investment. Many issues must be considered to obtain maximum benefit from the database investment.

7-2. Database Development

Following a complete requirements analysis, it is possible to determine the optimum strategy for obtaining the database. Two basic methods are available - acquiring an existing government or commercial database or building a new database.

a. Acquiring an existing database. The least expensive method of obtaining a database is nearly always to acquire an existing database, if one exists that will satisfy the users' needs. Potential data sources of information of available databases include:

- Federal Geographic Data Committee (FGDC) Manual of Federal Geographic Data Products - a compendium of databases available from 21 U.S. Government agencies, including Forest Service, Bureau of the Census, Defense Mapping Agency (DMA), National Park Service, U.S. Geological Survey (USGS), Federal Highway Administration, and National Aeronautics and Space Administration (NASA).
- National Geospatial Data Clearinghouse - an electronic clearinghouse for the NSDI, under development by the FGDC.
- State Geographic Information Activities Compendium - a hardcopy summary of GIS efforts by the State governments.
- Other USACE districts - other districts may have developed a database that would satisfy users' needs.
- Local governments - county and city governments maintain geospatial databases for planning and assessments.
- Commercial sources - many commercial companies offer digital geospatial data for sale.

If an existing database can be found that meets a portion of the requirements, consideration should be given to acquire and build upon it. This technique, while not always practical, can often yield substantial savings.

b. Building a New Database. If an acceptable database is not available from any known source, it is necessary to build a new database. If possible, the cost should be shared by another organization with similar data needs. In any case, the database must be designed in detail to meet all user's needs and meet all applicable standards.

(1) Database standards. The following standards must be used in the database design:

Data Format	FIPSPUB 173 Spatial Data Transfer Standard
Metadata	FGDC Content Standard for Digital Geospatial Metadata
Data Collection	USACE Interim Standards and Specifications for Surveys, Maps, Engineering Drawings, and Related Spatial Data Products
Data Accuracy	USACE Interim Standards and Specifications for Surveys, Maps, Engineering Drawings, and Related Spatial Data Products
Data Content	Tri-Service GIS/Spatial Data Standards
Data Symbology	Tri-Service GIS/Spatial Data Standards

(2) Database specification. A specification serves two purposes: (1) it provides a firm set of rules for data collection and database construction, and (2) it describes the database in sufficient detail to permit application development. This document will permit use of the database inside and outside of the producing organization and result in a substantial cost savings to users. The specification may take several forms, but at a minimum should include the following sections:

- Scope - a concise abstract of the coverage of the specification.

- Applicable Documents - a bibliographic listing of the standards and references used in developing the specification.

- Database Description - a summary of the information contained in and the structure/format of the database and the intended use of the data.

- Metadata - a listing of the static metadata elements, including accuracy, datum, scale/resolution, source, and projection (if applicable).

- Data Format - a detailed description of the data format.

- Data Dictionary - a dictionary of the feature and attribute codes used in the database.

(3) Database construction. The database is built to the meet the requirements of the Database Specification. It is advisable to prototype a database and distribute the prototype to potential users, along with a copy of the draft specification, prior to finalizing the design. This procedure is easier if the database is only for internal use, but no less valuable.

7-3. Data Documentation

Consistent, complete, current documentation of geospatial data is essential to maintaining the data investment and to reduce data duplication. Geospatial data should be documented using standard metadata formats as required by ER 1110-1-8156 and described in Chapter 8 of this manual. Metadata shall be developed using the "Content Standards for Geospatial Metadata." The following are general data documentation guidelines:

- Consider the value of the data set and develop metadata accordingly. For example, do not spend $2,000 to develop a metadata file for a $100 data set and do not spend $100 on metadata for a unique, expensive, and often used data set.

- At a minimum, data providers must make metadata files at the end of a project or data collection effort. This is the most logical approach for data collection efforts that are relatively brief from start to finish. For lengthier data collection efforts and projects, data providers should consider developing metadata files at the beginning of the effort as well, to ensure optimal data sharing and partnering.

- When deciding to develop metadata files for an entire data set vs. making metadata files per coverage or themes within a data set consider cost to develop the metadata, value of the data set, frequency of use of the data set, diversity of data characteristics between themes, etc.

- Find the appropriate metadata element to record important information about a data set. For example, the Metadata Standard does not include elements that are specifically labeled, "Modeling parameters" but there are many opportunities for the modeling community to accurately describe what parameters were used to develop the data set.

7-4. Quality Assurance

a. The primary goal of data quality assurance (QA) is to ensure a consistent and measurable accuracy throughout the database. Consistency is achieved through the use of documented, approved production procedures. Following production, an assessment of the quality of the data set should be made to ensure that the expected result was achieved.

b. The level of production control and the rigor with which the assessments must be made will vary among databases, and should be consistent with the requirements for the database. For example, a cadastral database will generally have exacting accuracy requirements and equally stringent requirements for consistency. This type of database will need to have detailed procedural documentation, a completion signature for each production step, and a comprehensive assessment of accuracy - significantly increasing the cost of production. Conversely, a small-scale database intended only as a background map for geographic orientation (e.g., Digital Chart of the World from DMA) will have much less stringent production documentation requirements and only a cursory accuracy assessment. The method used to measure accuracy can have an impact on the result, so this quality assessment should be made using standard measurement techniques, such as those described in the National Map Accuracy Standard, or local techniques that are well documented.

7-5. Data Access

a. Data and metadata produced by USACE, including those produced by commercial firms under contract to USACE, shall be made available to the public to the extent permitted by law, current policies, and relevant OMB policies, including OMB Circular No. A-130, "Management of Federal Information Resources." HQUSACE, with the assistance of the GD&S Field Advisory Group, has imple

mented a procedure to provide public access to USACE geospatial metadata through the National Geospatial Data Clearinghouse. This procedure is described in ER 1110-1-8156 and in Chapter 8 of this manual.

b. Each Command is responsible for establishing procedures for responding to requests from the public for geospatial data. The mechanics of ensuring public access to data holdings should be optimized for the existing organization and unique missions of each USACE Command. Commands may choose to have all requests for geospatial managed through a single office. Others may choose to have internal divisions respond to requests for the data they collect or produce.

7-6. Data Archive

a. Geospatial data represents a significant national asset. USACE Commands shall protect against the permanent loss of data by establishing an effective data archive. The archive shall contain a copy of all data sets produced within USACE, either in-house or on contract, and have an effective cataloging system such that data sets may be retrieved in reasonable time. The data archiving process (manual, automatic, or a combination) and frequency shall be appropriate for the application and sensitivity of the data.

b. The FGDC Historical Data Working Group has developed a draft brochure that provides guidance on the responsibilities of geospatial data developers and custodians. It lists 12 circumstances under which geospatial data sets should be archived. Any geospatial database that has current or potential future value to your Command or another Government agency that cannot be easily replicated must be considered for archive. This guidance has the effect of including nearly all geospatial data.

7-7. Data Maintenance

a. Data shall be maintained as needed to support USACE applications. As a data set is updated, its metadata shall also be updated and made available to the Clearinghouse.

b. It is recommended that the update cycle be determined during the requirements analysis based on currency requirements and budgetary constraints. Data maintenance can be a costly - but necessary - on going GD&S expense. Data maintenance is a cost multiplier that must be considered as part of the overall GD&S expense.

7-8. Data Liability

Data liability is an issue that requires more attention by legal experts. Liability for the data produced by USACE can come in two forms: (1) liability for data that does not meet its stated accuracy, and (2) liability for the unintended usage of the data [Aronoff, 1989].

a. Liability for Incorrect Data. The Federal Government is protected from being sued for providing "misinformation" under the Federal Tort Claims Acts. However, the government is not protected from "malpractice." There are few precedents in this area, but the best solution for USACE Commands is to develop sound procedures for data collection, handling, and processing and to adhere to the procedures. No USACE Command shall knowingly provide data that does not meet its stated accuracy nor which has undocumented or incorrect lineage. Every effort must be made to ensure that users understand the capabilities and limits of the data set they are using.

b. Liability for misuse. In "Geographic Information Systems: A Management Perspective," Stan Aronoff provides examples of how advanced GD&S can be employed to misuse public data in a manner that would be not be possible using hardcopy. There are no standing legal precedents in this area, so a USACE-wide policy on restricting access to certain types of data cannot yet be developed. It is important that all data provided through the Clearinghouse be properly documented as to its intended use, as required by the metadata standard.

7-9. Data Policies and Coordination

a. Policies. Each USACE Command may develop tailored GD&S policies to supplement and implement this guidance document. Tailored policies regarding GD&S technologies shall be drafted by the GD&S Technical Committee and approved by the GD&S Oversight Committee. Tailored policies shall adhere to the requirements of this document and all applicable standards, orders, and OMB circulars, and they shall support the goals of the NSDI.

b. Coordination of GD&S efforts. Coordination and prioritization of geospatial data acquisition and GD&S development efforts within a USACE Command shall be the function of the GD&S Technical Committee.

c. Coordination with authorities. The GD&S Technical Committee shall appoint a representative to coordinate USACE geospatial data acquisition and GD&S development efforts with local and state governments and national GIS coordinating committees. This representative may be the Command POC or another member of the Technical Committee. If it is necessary, multiple members of this committee can liaise outside of the Command; however, information exchange then becomes critical. The purpose of

the coordination is to reduce duplicative data collection efforts and identify cost sharing opportunities.

7-10. Importance of Geospatial Data Standards

Technical progress in GD&S has resulted in wide use by many organizations of geospatial data. GD&S users need geospatial data standards to better manage this data. Users should recognize that standards are supportive of efforts to reduce redundant data, make systems more efficient and lower project costs. To this end most standards provide flexibility which allows users to adapt the standards to their specific environment. This flexibility should be used with caution, however, to avoid distortion of the standards intent.

a. Benefits of geospatial data standards. The benefits of geospatial data standards come from making the activities which the geospatial data standards support more successful. As a suite of geospatial data standards is adopted the following benefits should accrue to the activities in an organization:

- There is a removal of barriers to geospatial data exchange and a more ordered and cost-effective data sharing as standards provide exchange mechanisms for the transfer of geospatial data between dissimilar systems.

- Geospatial data quality improves and configuration management of data increases as standards provide metadata to help organize and maintain the organizations internal spatial data.

- User confidence increases that geospatial data products are as advertised and providers of data can offer a warranty for data provided as products produced to a standard give users knowledge about the structure and content of data before acquiring it.

- Standards allow increased access to geospatial data. As a result new uses are found for the data as there is an increase in the number of geospatial data product choices available to the user community.

- Integration of systems is encouraged as geospatial data can flow between them, thus maximizing effective use of systems.

- Data collection duplication is reduced and investments in those geospatial data collected provide more return on the investment in them.

- Public access to geospatial data is improved and there is an increase in the GD&S user base due to data availability with an attendant diffusion of knowledge.

b. Types of geospatial data standards. Standards may be catalogued in several ways. One is by source of authority. Standards can be de facto, such as AutoCAD DXF, where the user community, through constant use adopts a practice without any formal certification. Standards may also be certified by a government body or a professional organization. Among these are the International Standards Organization (ISO), American National Standards Institute (ANSI), National Institute of Standards and Technology (NIST), the Federal Geographic Data Committee (FGDC) and the Tri-Service CADD/GIS Technology Center or those promulgated by professional organizations such as the American Congress on Surveying and Mapping (ACSM).

Another way to catalog standards is by the functionality the standard addresses. For the GD&S area these form a framework of standards as seen below:

- Hardware and Physical Connection Standards.

  These are standards that pertain to the physical connection and cabling of hardware devices.

- Application Standards.

  These are standards that impact the actual presentation and display of data in a GD&S, such as map design criteria.

- Software Standards.

  These are standards that address the development of software and software documentation including macros.

- Professional Standards.

  These are standards that establish levels of competency and training.

- Network Communication Standards.

  These are standards that address the protocols for the transfer of data and information from one computer system to another

Data Standards.

These are standards that address geospatial data transfer formats, accuracy, documentation, structure, content and management. It is these standards which are discussed below.

7-11. Authority for Geospatial Data Standards

Standards for geospatial data in USACE are governed by the following organizations.

a. Federal Geographic Data Committee. OMB Circular A-16 (Coordination of Surveying, Mapping and Related Spatial Data Activities) establishes a process to foster the development of a national spatial data framework for an information-based society with the participation of Federal, state, and local governments, and the private sector, and to reduce duplication of effort. It addresses the responsibilities of Federal agencies in the coordination of surveying, mapping, and related spatial data. It also establishes an interagency coordinating committee known as the Federal Geographic Data Committee (FGDC). The objective of the FGDC is to promote the coordinated development, use, sharing, and dissemination of surveying, mapping, and related geospatial data.

Executive Order 12906 Coordinating Geographic Data Acquisition and Access: The National Spatial Data Infrastructure (NSDI) states, among other things, that Federal agencies collecting or producing geospatial data shall ensure that data will be collected in a manner that meets all relevant standards adopted through the FGDC process. It also establishes the FGDC's authority over the NSDI and the National Geospatial Data Clearinghouse(Clearinghouse).

The FGDC can be contacted at:

U.S. Geological Survey
590 National Center
Reston, Virginia 22092
Telephone: (703) 648-4533
Fax: (703) 648-5755
Internet: gdc@usgs.gov

b. National Institute of Standards and Technology. The Computer Systems Laboratory at the National Institute of Standards and Technology (NIST) is responsible for developing technical, management, physical and administrative standards and guidelines for computer and related telecommunication systems. These standards are known as Federal Information Processing Standards (FIPS) Publications (FIPSPUBS). Many of these are of importance to GD&S and are discussed in the section on mandatory standards. FIPSPUBS are available from:

National Technical Information Service
5285 Port Royal Road
Springfield, VA 22161
Telephone: (703) 487-4650

c. Tri-Service CADD/GIS Technology Center. The Tri-Service CADD/GIS Technology Center is a multi-service vehicle to set standards and coordinate facilities CADD and GIS within the Department of Defense. It also promotes system integration and standards including naming conventions, common GIS layers, standard symbology, databases and analysis tools. The Tri-Service Center can be reached at:

Tri-Service CADD/GIS Technology Center
Information Technology Laboratory
(CEWES-ID-C)
U.S. Army Engineer Waterways Experiment Station
3909 Halls Ferry Road
Vicksburg, MS 39180-6199
Telephone: (601) 634-4582
Fax: (601) 634-4584
Internet: wesimda@ex1.wes.army.mil

7-12. Applicable Standards

Some standards are going to impact all USACE organizations. Others will only be of concern to USACE organizations with special circumstances. Mandatory standards are those that are sufficiently mature that all USACE components must follow them. ER 1110-1-8156, paragraph 6 requires that anyone who believes these standards are inappropriate for their use must apply to CECW-EP-S for a waiver. The waiver must explain why the standards are inappropriate and what will be used instead. Recommended standards are those where compliance is encouraged but the maturity of the standard is not sufficient for them to be mandatory.

a. Mandatory geospatial data standards.

(1) Federal Information Processing Standards Publications.

Use of Federal Information Processing Standards (FIPS) are mandatory for USACE GD&S. The USGS Open-file Report 88-105, *A Process for Evaluating GIS,* provides a 63 page annotated list of Federal Information Processing Standards (FIPS) and National Bureau of Standards GD&S related standards, guidelines and references. The report can be acquired from:

USGS Earth Science Information Center
507 National Center
Reston, VA 22092
Toll Free Number: 1-800-USA-MAPS
Telephone: (703) 648-5920
FAX: (703) 648-5548

The FIPS Publications are available from:

National Technical Information Service
Computer Products Office
5285 Port Royal Road
Springfield, VA 22161
Telephone: (703) 487-4600

(2) FIPS 173 Spatial Data Transfer Standard.

The Spatial Data Transfer Standard (SDTS) provides specifications for the organization and structure of digital spatial data transfer, definition of spatial features and attributes, and data transfer encoding. The purpose of the standard is to promote and facilitate the transfer of digital spatial data between dissimilar computer systems. This standard is for use in the acquisition and development of government applications and programs involving the transfer of digital spatial data between dissimilar computer systems. The use applies when the transfer of digital spatial data occurs or is likely to occur within and/or outside of the Federal Government. It is likely that vendors/producers of leading GD&S will introduce SDTS import/export capability in the future. The SDTS is a key element of the NSDI and is the result of FGDC efforts.

(3) Content Standards for Digital Geospatial Metadata.

This standard specifies the information content of metadata for a set of digital geospatial data. The purpose of the standard is to provide a common set of terminology and definitions for concepts related to these metadata. This standard is the data documentation standard referenced in Executive Order 12906 which mandates the documentation of all new geospatial data starting 11 January 1995 and the development of a plan to document geospatial data previously collected or produced, by 11 April 1995.

The metadata standard is the product of the FGDC. Executive Order 12906 instructs Federal agencies to use the metadata standard to document new geospatial data beginning in 1995 and to provide these metadata to the public through the National Geospatial Data Clearinghouse. The GD&S vendor community may provide metadata software in the future. Several pieces of public domain metadata software are available. Document.aml is a public domain metadata macro for use with ARC/INFO.

CORPSMET is a DOS-based public domain metadata software developed by USACE. CORPSMET can be used to document any geospatial data independently of the geospatial data system in use.

(4) USACE Interim Standards and Specifications for Surveys, Maps, Engineering Drawings, and Related Spatial Data Products.

This is to be used for prescribing standards and specifications for USACE field surveys, maps, engineering drawings, and related spatial data products. It is applicable to all HQUSACE elements, major subordinate commands, districts, laboratories, and field operating activities having civil works, military programs, and environmental restoration responsibilities. It also applies to functional areas having responsibilities for regulatory investigations and studies, real estate, and support to Army installation master planning, and other functions involving surveying, mapping, or spatial database development.

(5) Tri-Service GIS/Spatial Data Standards (TSSDS).

These standards are applicable to all Department of Defense activities having civil works or public works, military programs, and environmental programs or that are responsible for facilities/installation management. They prescribe standards and specifications for GIS and related spatial data. The intent is to create standards that will satisfy the project life-cycle concept for digital data. The TSSDS are intended to contain requirements for standard data entry, storage and retrieval, using predefined screen displays and plotting routines. There are many subcommittees and working groups of the FGDC that are development of content standards and the work is at various levels of maturity. The final versions of these standards will be incorporated into the TSSDS for distribution and use throughout USACE therefore by using the most recent version of the TSSDS one will also be using the most recent FGDC content standards. The proponent for this standard is the Tri-Service CADD/GIS Technology Center.

b. Military standards. Those elements of USACE working with Defense Mapping Agency (DMA) geospatial data may find it necessary to use Military Standards (MIL-STD). As of 27 December 1994 DoD agencies may not require vendors to support Military Standards unless the agency obtains a waiver to do so. USACE employees should check with Procurement or Engineering Chiefs for the most current guidance related to this requirement. The DMA standards and specifications program is described in Digitizing the Future. It is available from the DMA Headquarters Plans and Requirements Directorate in care of: